EPA/600/R-99/029

Innovative Urban Wet-Weather Flow Management Systems

By

James P. Heaney
Department of Civil, Environmental, and Architectural Engineering
University of Colorado
Boulder, CO 80309

Robert Pitt
Department of Civil and Environmental Engineering
The University of Alabama at Birmingham
Birmingham, AL 35294

and

Richard Field
Water Supply and Water Resources Division
National Risk Management Research Laboratory
Edison, NJ 08837

Cooperative Agreement Nos. CX824932 & CX 824933

Project Officer

Chi-Yuan Fan
Water Supply and Water Resources Division
National Risk Management Research Laboratory
Edison, NJ 08837

NATIONAL RISK MANAGEMENT RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OH 45268

Notice

The information in this document has been funded wholly or in part by the United States Environmental Protection Agency under cooperative agreements no. CX824932 for the American Society of Civil Engineers and no. CX 824933-01-0 for the University of Alabama at Birmingham. Although it has been subjected to the Agency's peer and administrative review and has been approved for publication as an EPA document, it does not necessarily reflect the views of the Agency and no official endorsement should be inferred. Also, the mention of trade names or commercial products does not imply endorsement by the United States government.

Foreword

The U.S. Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water resources. Under a mandate of national environmental laws, the Agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. To meet this mandate, EPA's research program is providing data and technical support for solving environmental problems today and building a science knowledge base necessary to manage our ecological resources wisely, understand how pollutants affect our health, and prevent or reduce environmental risks in the future.

The National Risk Management Research Laboratory is the Agency's center for investigation of technological and management approaches for reducing risks from threats to human health and the environment. The focus of the Laboratory's research program is on methods for the prevention and control of pollution to air, land, water and subsurface resources; protection of water quality in public water systems; remediation of contaminated sites and ground water; and prevention and control of indoor air pollution. The goal of this research effort is to catalyze development and implementation of innovative, cost-effective environmental technologies; develop scientific and engineering information needed by EPA to support regulatory and policy decisions; and provide technical support and information transfer to ensure effective implementation of environmental regulations and strategies.

This publication has been produced as part of the Laboratory's strategic long-term research plan. It is published and made available by EPA's Office of Research and Development to assist the user community and to link researchers with their clients.

E. Timothy Oppelt, Director
National Risk Management Research Laboratory

Abstract

This research project describes innovative methods to develop improved wet weather flow (WWF) management systems for urban developments of the 21st century. This document addresses the competing objectives of providing drainage services at the same time as decreasing stormwater pollutant discharges. Water quality aspects of WWF discharges and associated receiving water problems have only been studied for a relatively short period (a few decades), compared to conventional drainage designs (a few centuries), and few large-scale drainage systems adequately address both of these suitable objectives.

General principles of urban water management are presented that might permit the development of more sustainable systems by integrating the traditionally separate functions of providing water supply, collecting, treating, and disposing of wastewater, and handling urban WWF. Integration can be achieved by designing neighborhood scale, integrated infrastructure systems wherein treated wastewater and stormwater are reused for nonpotable purposes such as lawn watering and toilet flushing. The automobile is seen to have caused major changes in urban land use in the 20th century. For the average urban family, the area devoted to streets and parking in their neighborhood exceeds the area devoted to living. Similarly, more area is devoted to parking than to office and commercial space in urban areas. The net result of the large scale changes to accommodate the automobile in cities is about a two to three fold increase in impervious area per family and business activity.

The physical, chemical, and biological water quality characteristics of urban runoff are evaluated and summarized. Then, the impacts of urban WWF on receiving waters are evaluated. These impacts on surface and groundwater are complex and difficult to evaluate. Physical changes in smaller urban streams can be detected in terms of degraded channels from higher peak flows. Also, sediment transport characteristics change with urbanization. Toxic effects on aquatic organisms have been detected.

Traditionally, wet-weather collection systems were designed to move stormwater from the urban area as quickly as possible. This design approach often simply transferred the problem from upstream to downstream areas. More recently, restrictions on the allowable maximum rate of runoff have forced developing areas to include onsite storage in detention ponds to control these peak rates of runoff. On-site detention also allows smaller pipe sizes downstream. In the early part of the 20th century, communities relied on combined sewers. Later, separate storm and sanitary sewers became accepted practice. However, as the need to treat more contaminated storm water becomes more apparent, it is necessary to take a fresh look at combined sewers. However, because of the strong trend to lower density urban development to accommodate the automobile, the quantity of urban runoff per family is two to three times what it was with higher density developments. Most of the traffic flow in cities occurs on a relatively small percentage of streets, about 10-20%. Also, most parking areas are underutilized. Thus, it may be possible to focus WWF treatment on these

more intensively used areas including commercial and industrial areas. This finding suggests that hybrid collection systems may be attractive alternatives for 21^{st} century collection systems. Another innovative option is to oversize sewer systems and utilize storage in the sewers as part of a real-time control system.

Extensive discussions regarding the effectiveness of a wide variety of WWF controls are presented in two chapters. These descriptions include design guidelines. Source controls as well as downstream controls are included. Source area controls, especially biofiltration practices that can be easily implemented with simple grading, may be appropriate in newly developing areas. In addition, critical source areas (such as vehicle service facilities) may require more extensive onsite treatment strategies. An innovative approach is to reuse stormwater within the same service areas for irrigation, toilet flushing, and other nonpotable purposes. More aggressive stormwater reuse systems would capture roof runoff in cisterns, treat this water, and use it for potable purposes. Monthly water budgets for cities throughout the United States indicates that sufficient quantities of precipitation are generated, except in the arid southwestern United States, to make such systems technically feasible. The cost of providing for water infrastructure is summarized. The traditional problem of finding the optimal size of service area for water supply is addressed by finding the minimum sum of the costs of source acquisition, treatment, and distribution. For wastewater and stormwater, the minimum total cost is the sum of collection, treatment, and disposal. These costs per residence have grown substantially as development densities have decreased. Also, if wastewater and stormwater reuse are included, then the optimal size of infrastructure system may be at the neighborhood scale since piping costs remain the largest single cost in urban water infrastructure.

Lastly, institutional arrangements need to change in order to successfully implement changes in how urban water infrastructure is managed. Privatization, moving from large centralizes systems to neighborhood based systems, and other projected changes required innovative changes in the governing institutions.

This report was submitted in fulfillment of Cooperative Agreement Nos. CX824932 and CX824933 by American Society of Civil Engineers (University of Colorado and the Urban Water Resources Research Council) and the University of Alabama at Birmingham, respectively under the sponsorship of the U.S. Environmental Protection Agency. This report covers a period from May 1996 to August 1998, and work was completed as of August 1998.

Contents

Tables

Figures

Abbreviations and Acronyms

A	Area
AASHTO	Association of State Highway and Transportation Officials
ac-ft	Acre-foot
ADT	Average daily traffic
AMSA	Association of Metropolitan Sewerage Agencies
APWA	American Public Works Association
ASCE	American Society of Civil Engineers
AWRA	American Water Resources Association
AWWA	American Water Works Association
AWWARF	American Water Works Association Research Foundation
BASINS	Better Assessment Science Integration Point and Nonpoint Sources
BCW	Boulder Creek Watershed
BMP	Best management practice
BOD	Biochemical oxygen demand
C	Runoff coefficient (in Rational method)
C of V	Coefficient of variation (standard deviation/mean)
CCA	Copper, chromium, arsenic
COD	Chemical oxygen demand
CSO	Combined sewer overflow
CY	Calendar year
DBO	Design-build-operate
DCIA	Directly connected impervious area (See IA)
DSS	Decision support systems
DU	Dwelling unit
DUD	Dwelling unit density
DWF	Dry weather flow
EPA	U.S. Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FHA	Federal Housing Administration
FHWA	Federal Highway Administration
fps	Feet per second
ET	Evapotranspiration
gpcd	Gallons per capita per day
gpd/idm	Gallons per day per inch diameter per mile
GIS	Geographic information system
ha	Hectare
HCR	High rain intensity, Clean, and Rough street
HCS	High rain intensity, Clean, and Smooth street
HDR	High rain intensity, Dirty, and Rough street
HDS	High rain intensity, Dirty, and Smooth street
HOV	High occupancy vehicle
HUD	U.S. Department of Housing and Urban Development
I	Imperviousness

IA	Impervious area (See DCIA)
IBDU	Isobutylidene diurea
I/I	Infiltration and/or inflow
ITE	Institute of Transportation Engineers
ISS	Integrated storm-sanitary system
J	Julian day number (e.g., J=365 for December 31)
kl	Kiloliter
l	Liter
L	Length of street per dwelling unit
lb/ft^2	Pound per square foot
LCE	Life-cycle engineering
LCR	Light rain intensity, Clean, and Rough street
LCS	Light rain intensity, Clean and Smooth street
LDR	Light rain intensity, Dirty, and Rough street
LPS	Low pressure sewers
m	Meter
MCTT	Multi-chambered treatment train
mgd	Million gallons per day
ml	Milliliter
mm	Millimeter
MMI	Man-machine interface
MTBE	Methyl-tert-butyl ether
MTBSC	Mean time between service calls
MVS	Modern vacuum system
N/m^2	Neuton per square meter
NAREUS	North American End Use Study
NCRS	National Resource Conservation Service (formerly, SCS, Soil Conservation Service)
NMC	Nine minimum controls
NPDES	National Pollution Discharge Elimination System
NPS	Non-point source
NSF	National Science Foundation
NURP	Nationwide Urban Runoff Program
NWS	National Weather Service
O&M	Operation and maintenance
OIA	Other impervious area
OWRR	Office of Water Resources Research
P	Precipitation (inches)
PAH	Polycyclic aromatic hydrocarbons
PD	Population density
PET	Potential evapotranspiration
POC	Purgable organic carbon
PSCO	Public Service Company of Colorado
R	Runoff volume
RCRA	Resource Conservation and Recovery Act
ROW	Right of way

RPE	Runoff producing event
RTC	Real time control
SCADA	Supervisory control and data acquisition
SCS	Soil Conservation Service (now the NRCS, National Resource Conservation Service)
SDC	System development charges
SDGS	Small diameter gravity sewer
SOV	Single occupancy vehicle
STD	Standard deviation
STEP	Septic tank effluent pumping
SS	Suspended solids
SSES	Sewer System Evaluation Survey
SSO	Sanitary sewer overflow
STORM	Storage, Treatment, Overflow and Runoff Model
THM	Trialomethane
TND	Traditional neighborhood development
TOC	Total organic carbon
TSS	Total suspended solids
µm	Micrometer
UF	Urea formaldehyde
ULI	Urban Land Institute
USEPA	U.S. Environmental Protection Agency
USGS	U.S. Geological Survey
UV	Ultraviolet
UWRRC	Urban Water Resources Research Council (of ASCE)
VMT	Vehicle miles traveled
VOC	Volatile organic compound
WARMF	Watershed Analysis Risk Management Framework
WEF	Water Environment Federation
WET	Whole effluent toxicity
WSIUA	Water sustainability in urban areas
WWF	Wet weather flow

Acknowledgments

This document was prepared for the U.S. Environmental Protection Agency (EPA) under Cooperative Agreement Nos. CX824932 and CX824933. The support of the project by the EPA Offices of Water, Wastewater Management, and Research and Development; the EPA National Risk Management Research Laboratory; the Urban Water Resources Research Council of the American Society of Civil Engineers; and the University of Alabama at Birmingham is acknowledged and appreciated.

Stuart G. Walesh, Ph.D., P.E., an independent consultant, peer-reviewed and edited this report. Peer review comments were also provided by Professor Richard M. Ashley, Ph.D. of the University of Abertlay Dundee, Scotland. The cooperation and helpful suggestions provided by Chi-Yuan Fan, P.E., DEE, Project Officer, is acknowledged. Furthermore, the contributions of all of the following individuals are acknowledged and appreciated:

Ted Brown, Wright Water Engineers, Inc. Denver, CO.

Jane Clary, Wright Water Engineers, Inc., Denver, CO.

Richard Field, P.E., U.S. Environmental Protection Agency, National Risk Management Research Laboratory, Water Supply and Water Resources Division, Edison, NJ.

James P. Heaney, Ph.D., Department of Civil, Environmental, and Architectural Engineering, University of Colorado at Boulder, Boulder, CO.

Jonathan Jones, P.E., Wright Water Engineers, Inc., Denver, CO.

David Sample, Department of Civil, Environmental, and Architectural Engineering, University of Colorado at Boulder, Boulder, CO.

Brian W. Mack, P.E., Camp Dresser and McKee Inc., Maitland, FL.

Robert Pitt, Ph.D., P.E., DEE, Department of Civil and Environmental Engineering, The University of Alabama at Birmingham, Birmingham, AL.

Michael F. Schmidt, P.E., Camp Dresser and McKee Inc., Maitland, FL.

Michelle Solberg, Camp Dresser and McKee Inc., Maitland, FL.
Ben Urbonas, P.E., Denver Urban Drainage and Flood Co/ntrol District, Denver, CO.

Len Wright, Department of Civil, Environmental, and Architectural Engineering, University of Colorado at Boulder, Boulder, CO.

In addition, Melissa Lilburn, graduate student at the University of Alabama at Birmingham and Steve Burian, graduate student at the University of Alabama, along with Stephan Nix, Ph.D. and Rocky Durrans, Ph.D. professors at the University of Alabama, contribution to chapters 4 and 5 are also acknowledged and appreciated. Helen Egidio assisted to finalize the manuscript, and Stephanae Liik diligently restored and developed an electronic version of the most complex figures.

It is acknowledged and appreciated the quick and meticulous work of the City of Boulder Open Space GIS Laboratory and Hydrosphere, both of whom replaced severely damaged figures.

Chapter 1

Introduction

James P. Heaney, Robert Pitt, and Richard Field

Introduction

Stormwater has traditionally been considered a nuisance, requiring rapid and complete drainage from areas of habitation. Unfortunately, this approach has caused severe alterations in the hydrological cycle in urban areas with attendant, mostly negative, changes in receiving water conditions and uses. This historical “water as a common enemy” approach has radically affected the way urban dwellers relate to water. For example, most residents are not willing to accept standing water near their homes for significant periods of time after rain has stopped.

However, a new, innovative approach to stormwater management is beginning to appear. There are many examples where engineers, planners, landscape architects and others have successfully integrated water into the urban landscape. In many cases, water has been used as a focal point in revitalizing downtown areas. Similarly, many arid areas are looking at stormwater as a potentially valuable resource, with stormwater being used for on-site beneficial uses, instead of being quickly discharged as a waste.

New actual and potential innovative approaches to stormwater management are described in this report. Overviews of individual chapters are presented below.

Chapter 2: Principles of Integrated Urban Water Management

The purpose of this chapter is to review the literature on innovative urban developments, in general, evaluate principles of sustainability, and present the urban stormwater management problem within this broader context. The focus of this report is new urban developments and these developments are at the neighborhood scale. Control methods include source controls at the individual parcel level.

Trends in urbanization during the 20th century are described including the impact of the automobile and subdivision regulations. Urban sprawl has often been the result of such changes. Possible emerging land use forms are described that might be more sustainable than present systems. Issues are presented to help decide whether smaller or larger scale infrastructure systems are preferable. Finally, the sources of runoff in urban areas are described along with a description of their relative importance.

Chapter 3: Sustainable Urban Water Management

Water supply, wastewater, and stormwater systems are explored in this chapter, first individually and then in an integrative manner. Key areas of potential integration of these three functions are reuse of wastewater and stormwater to reduce the required net import of water for water supply. The literature review summarizes previous and on-going work nationally and internationally to develop more sustainable urban water management systems. A systems view of urban water management was first advocated in the late 1960's. This approach is summarized. Principles for sustainable urban water infrastructure systems are presented.

Urban water budgets provide a way to evaluate the relative importance of the various components of the urban water system. The results of a recently completed national residential water use study are described along with the results of several water budget studies from Europe and Australia. Then, monthly water budgets for Denver, CO and New York City, NY are presented. Lastly, some alternative future urban water scenarios are described ranging from the status quo to aggressive water conservation and reuse programs.

Chapter 4: Source Characterization

The sources of the stormwater pollutants and flows that are likely to be preventing beneficial uses must be recognized and quantified before an effective stormwater management strategy can be implemented. This chapter gives an overview of the obvious stormwater pollutant sources in urban areas, especially natural sources (soils, atmospheric dustfall, and rain) and the washoff of contaminated dirt from pavements (the most popular location for source control efforts). Included in Chapter 4 are summaries of actual sheetflow runoff quality obtained during rains from numerous source areas (roofs, landscaped areas, parking and storage areas, driveways, sidewalks, and streets) for commercial, industrial, and residential land use areas. The chapter concludes describing a study that investigated toxic heavy metal and organic pollutant sources. Information and ideas presented in this chapter can be used to identify significant sources of problem pollutants and understand how stormwater can be better controlled at critical source areas and/or at a downstream outfall.

Chapter 5: Receiving Water and Other Impacts

A critical element to be investigated as part of a stormwater management program is an understanding of the local receiving water problems. This chapter reviews many types of problems that have been identified and documented during studies throughout the country. The list of potential problems is diverse and long, although relatively few may be relevant for any given geographic area. Some of the most common types of receiving water problems that have been investigated relate to aquatic life uses. Numerous studies have compared aquatic life (usually fish and benthic macroinvertebrates) in urban streams with reference streams. Most of the investigations examined toxic pollutant causes of the noted aquatic organism differences, but recent investigations focused more on habitat issues caused by stormwater discharges (e.g., contaminated and fine-grained sediments, unstable streambeds, variable and high flows, and destruction of refuge areas).

Human health issues associated with stormwater discharges are also reviewed. Potential groundwater impacts caused by inadvertent and by designed subsurface disposal of stormwater are also examined. Chapter 5 includes emerging tools that many States are using to measure receiving water problems, especially bioassessment procedures that integrate numerous relatively inexpensive field measurement components.

Chapter 6: Collection Systems

Stormwater and other wastewater collection systems are a critical link in the urban water cycle, especially under wet-weather conditions. In the context of pollution control, these systems transport sanitary wastewater, stormwater, industrial wastewater, non-point source pollution, inflow, and infiltration. Understanding the problems associated with modern sewer collection systems is enhanced by reviewing the history of collection systems in the U.S. Problems associated with present day collection systems are described including the challenge of infiltration and inflow. The emerging issue of sanitary sewer overflows is discussed. The importance of understanding the nature of sewer solids is described with emphasis on the role of solids in determining sewer design criteria. Innovative sewer design and monitoring systems are discussed.

Chapter 7: Assessment of Stormwater Best Management Practice Technology

The use of stormwater controls to manage the quality and quantity of urban runoff has become widespread in the U.S. and in many other countries. As a group they have been labeled best management practices, or BMPs. Structural BMPs are designed to function without human intervention at the time wet weather flow is occurring, that is, they are expected to function unattended during a storm and to provide passive treatment. Nonstructural BMPs, as a group, are a set of practices and institutional arrangements, both with the intent of instituting good housekeeping measures that reduce or prevent pollutant deposition on the urban landscape.

Much is known about the technology behind these practices, much is still emerging and much remains yet to be learned. Many of these controls are used without full understanding of their limitations and their effectiveness under field conditions. Uncertainties in the state of practice associated with structural BMP selection, design, construction and use are further complicated by the stochastic nature of stormwater runoff and its variability with location and climate. Examination of precipitation records throughout the U.S. reveals that the majority of individual storms are relatively small, often producing less precipitation and runoff than used in the design of traditional storm drainage networks. Chapter 7 describes a number of structural and non-structural BMPs with emphasis on their effectiveness in removing pollutants and in mitigating flow rates. BMP effectiveness in addressing some of the impacts of urban runoff on receiving water systems is also discussed.

Chapter 8: Stormwater Storage-Treatment-Reuse Systems

The overall effectiveness of a variety of stormwater BMP's is evaluated in the previous chapter. Two other aspects of control of stormwater: high-rate treatment and the potential effectiveness of using stormwater for supplemental irrigation, are described in Chapter 8. Presented is a review of ways to evaluate the tradeoff between storage and treatment of wet-weather flows. Then the potential for high-rate operation of wastewater treatment plants during wet-weather periods is discussed. Stormwater reuse offers the possibility of significantly reducing water demand for irrigation and toilet flushing. The approximate size of on-site storage needed and how it varies with location is presented. A monthly water budget is used as part of this to estimate storage needs.

Chapter 9: Urban Stormwater and Watershed Management: A Case Study
Interest in watershed management has waxed and waned over the past century. During the 1980's, primary reliance was placed on a command and control approach for addressing water resources problems including stormwater. A strong move back to the watershed management approach began a few years ago. Watershed analysis and planning methodologies are reviewed.

A detailed case study of Boulder Creek Watershed (BCW) and Boulder, CO is presented. (This case study emphasizes the analysis aspect of urban stormwater and watershed management. Appendix A in this report is a case study that emphasizes the planning aspect of urban stormwater and watershed management). With the beginning of mining in 1858, the water and land associated with various forms of development had a significant impact on BCW. The watershed has been drastically altered by activities such as mining, urbanization, agriculture and hydropower development. BCW suffered serious early stormwater pollution from the original mining activities.

Thus, nonpoint pollution is an old problem in BCW. The watershed has also been adapted to provide water supply, flood control, recreation, and instream flow needs. The adaptations are both structural and nonstructural. Structural interventions include construction of reservoirs, canals, pipelines, pump stations, hydropower generation, water and wastewater collection and treatment systems, flood control levees, instream and wetland restoration, and imports and exports of water. Nonstructural interventions include flood warning systems, floodplain management, water rights enforcement, water conservation programs, and education about watershed protection.

The end result of all of these interventions is a complex watershed system that has been adapted to serve the needs of society as well as the natural system. This level of development and adaptation is typical of watersheds in the U.S. and other developed areas. Thus a watershed should be dealt with as a system in contrast with isolating system components and ignoring the system's complexity. While the focus of this report is urban stormwater quality management, these other considerations should also be borne in mind.

Chapter 10: Cost Analysis and Financing of Urban Water Infrastructure
This chapter summarizes water, wastewater, and stormwater infrastructure costs for cities in the U.S. While the main theme of this report is stormwater, some of the innovative ideas proposed would reuse stormwater for reducing water supply demands (e.g., for irrigation water). The effect of dwelling unit density on the demand for water infrastructure is presented. Previous efforts to find the optimum scale of urban water

systems are described. Summary cost functions for a variety of water resources facilities are presented.

Stable funding is an essential ingredient in developing and maintaining viable urban water organizations, whether they are stormwater utilities, watershed organizations, or other organizational forms. Integrated management offers the promise of improved economic efficiency and other benefits by combining multiple purposes and stakeholders. However, the benefits from integrated management exacerbate problems of financing these more complex organizations because ways must be found to assess each stakeholder's "fair share" of the cost of this operation. An overview of utility financing in the water, wastewater, and stormwater areas is presented.

Chapter 11: Institutional Arrangements
Stormwater Management Institutions of the 21st century face many challenges. Federal stormwater permitting requirements will affect most cities. Funding and staffing are likely to remain tight, even though stormwater regulations and requirements continue to expand. Stormwater management will be only one of a long list of issues that must be addressed by local governments. Given the time and budget constraints that staff will face, local governments will have to decide where stormwater management lies relative to other priorities. This is no easy task, given that the benefits of stormwater management can be elusive to quantify.

New stormwater management facilities must be financed and constructed. The public must be better educated on the significance of stormwater issues and stakeholders should be increasingly involved in urban water management. Research must lead to new technologies for treating and retaining stormwater runoff. Institutions will need to issue guidance on complicated and often controversial issues such as riparian corridor preservation, impervious area limitations, conservation easements, innovative zoning techniques and other subjects. Given these challenging tasks, Chapter 11 briefly characterizes the existing models of stormwater management institutions, identifies five essential characteristics of future stormwater management institutions, and describes specific technical and administrative issues that these stormwater management institutions must address.

Further, existing stormwater regulations are transitioning from the promulgation and implementation stages to the enforcement stage, where local governments may face legal challenges, particularly as a result of land use restrictions. Coordination among local, state, federal and private entities is and will continue to be a challenge. Stormwater management institutions have to address both water quality and quantity issues. In some cases, this will require retrofitting existing stormwater quantity structures to address stormwater quality issues and to improve their drainage and flood control function.

A planning case study to illustrate innovative stormwater management in new development is presented in **Appendix**. It is a condensed version of the Southeast

Annexation Area Lake Hart Basin Master Stormwater Management Plan (LHMSMP), City of Orlando, Orange County, FL. The general goals of the LHMSMP are the development of an integrated stormwater, wetland, and open space management system that would balance preservation of natural systems with land development. The general goals are to be accomplished by meeting the following three key objectives in a cost-effective manner: flood control, pollution control, and ecosystem management (which includes wetlands protection, aquifer recharge, and water conservation).

Chapter 2

Principles of Integrated Urban Water Management

James P. Heaney

Introduction

The purpose of this chapter is to review the literature on innovative urban developments, in general; evaluate principles of sustainability; and present the urban stormwater management problem within this broader context.

The Neighborhood Spatial Scale

The spatial scales for urban developments to be evaluated in this report are defined as follows:

1. Individual parcel: the smallest spatial scale consisting of an individual lot that may contain a house, apartment, commercial, industrial, or public activity.
2. Block: collection of parcels bounded by streets. For example, in higher density, older neighborhoods with gridiron streets, the typical area of a block is 1/8 x 1/16 of a mile or five acres. Blocks tend to be larger in area for contemporary lower density developments with block sizes being as large as 20 acres in size.
3. Subdivision: single land development, typically with the same land uses. Subdivisions are assumed to range in size from 25 to 100 acres.
4. Neighborhood: mixture of residential, commercial, public, and perhaps industrial land uses. The neighborhood is assumed to be an integrated, partially self-sustained, urban system. Typical sizes would be 100-1,000 acres.
5. New Town: cluster of neighborhoods designed to be largely self-sustaining in that the town provides sufficient employment opportunities for the local residents. Population sizes range from 20,000 to 60,000 people.

While the scope of this report are developments with populations less than 50,000 people, the area can either be greenfield (previously undeveloped land) or brownfield (urban redevelopment).

Trends in Urbanization

Historical Patterns

Certain background information helps to understand and evaluate future neighborhood stormwater systems. Examples are understanding historical land use patterns, factors stimulating changes in those land use patterns, and projecting expected future patterns of urban land use and the extent to which urban infrastructure might influence, or be influenced, by these changes.

Cities evolve in response to the inhabitants' needs for mutual self-protection, commerce, education, and cultural exchange. The late 1800's signaled the end of the "pioneer era" in the United States during which people migrated from place to place in search of a better way of life. For the first 20 years of the 20th century, infrastructure in cities focused on non-transportation related needs. However, the growing importance of the automobile, beginning in the 1920's, forced city managers to devote an increasing portion of their budgets to accommodating this new mode of transportation. Prior to World War II, U.S. cities developed around the concept of mixed neighborhoods as part of villages, towns, and cities. Beginning in the late 1940's, suburbia began to dominate urban America. Early suburbia had its origins in the late 19th century with urban dwellers seeking to escape the blighted conditions of cities. Suburban living in the late 19th century was made possible by commuter trains that provided reasonable access to cities from outlying areas.

Impact of the Automobile

The automobile is having a profound impact on urban developments during the 20th century. A summary of trends in population and automobile use in the United States from 1915 to 1994 is shown in Table 2-1. During this period, the U.S population grew by a factor of 2.6 from 100 to 261 million people and the number of automobiles grew by a factor of 80 from 2.5 million to nearly 200 million. The most dramatic growth in automobiles occurred since World War II. For example, from 1945 to 1955, the number of automobiles doubled from 31 million to 62.8 million. From 1955 to 1995, the number of automobiles tripled to over 200 million vehicles. The trends in growth of population and automobiles, shown in Figure 2-1, indicate that the rate of increase of vehicles is much greater than population growth.

The trend in vehicles per capita is shown in Figure 2-2. At present, there are 0.76 vehicles per capita. Perhaps, this is a saturation level based on the percentage of the population that is older than the minimum driving age. For example, 79.9% of the U.S. population is over 13 years old (National Safety Council 1995).

The vehicle miles traveled (VMT) per capita has continued to rise at a steady rate since 1945 as shown in Figure 2-3. Projections for the State of Colorado indicate that the 1995 VMT of 10,000 is expected to increase to 11,130 by the year 2020 (Yuhnke 1997). The average American drives twice as much as the average European or Japanese citizen (Kunstler 1996). Americans use cars for 82% of their trips compared to 48% for Germans, 47% for the French, and 45% for the British (Kunstler 1996). Between 1960 and 1990, Americans commuting by car increased from 69.5% to 86.5% while commuting by public transit decreased from 12.6% to 5.3% and walking decreased from 10.4% to 3.9% (Goldstein 1997).

With only 5% of the world's population, the United States consumes a quarter of the world's oil, half of which is used in motor vehicles (Kunstler 1996). Over 60,000 square miles of U.S. land is paved over which is 2% of the total surface area and 10% of the arable area (Kunstler 1996). The American public has subsidized this development through a combination of incentives such as large defense expenditures to protect oil producing countries, subsidized highway construction, and "free" parking. Auto tolls

Table 2-1. Changing patterns of automobile use in the U.S., 1915-1996 (Tetra Tech 1996).

Year	No. of Vehicles millions	No. of Drivers millions	Vehicle Miles/yr. Billions	Population millions	Drivers/ Population	Vehicles/ Population	Vehicles miles/ capita
1915	2.5	3.0		100	3.0%	0.03	
1920	9.2	14.0		107	13.1%	0.09	
1925	21.1	30.0	122	115	26.2%	0.18	1064
1930	26.7	40.0	206	123	32.5%	0.22	1672
1935	26.5	39.0	229	127	30.7%	0.21	1801
1940	32.5	48.0	302	132	36.3%	0.25	2285
1945	31.0	46.0	250	132	34.7%	0.23	1888
1950	49.2	62.2	458	151	41.2%	0.33	3030
1955	62.8	74.7	606	164	45.5%	0.38	3690
1960	74.5	87.4	719	180	48.6%	0.41	3997
1965	91.8	99.0	888	194	51.1%	0.47	4588
1970	111.2	111.5	1120	204	54.7%	0.55	5494
1975	137.9	129.8	1330	215	60.3%	0.64	6178
1980	161.6	145.3	1521	227	63.9%	0.71	6694
1985	177.1	156.9	1774	239	65.6%	0.74	7420
1990	192.9	167.0	2148	249	67.1%	0.77	8626
1994	199.4	175.1	2347	261	67.1%	0.76	8992

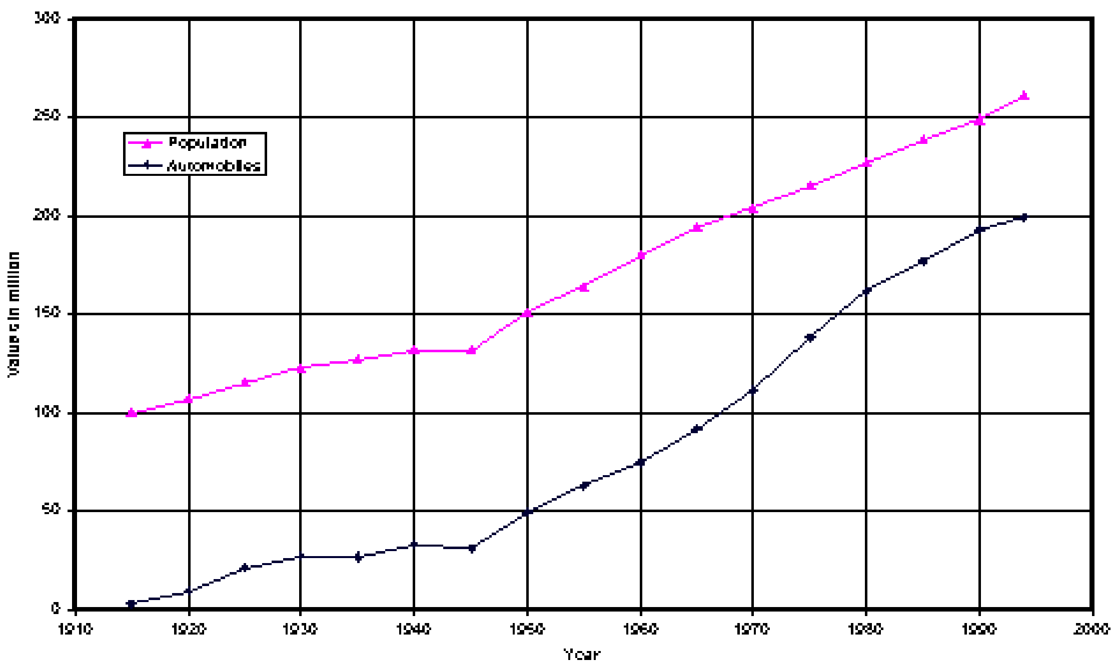


Figure 2-1. Trends in U.S. population and ownership of automobiles.

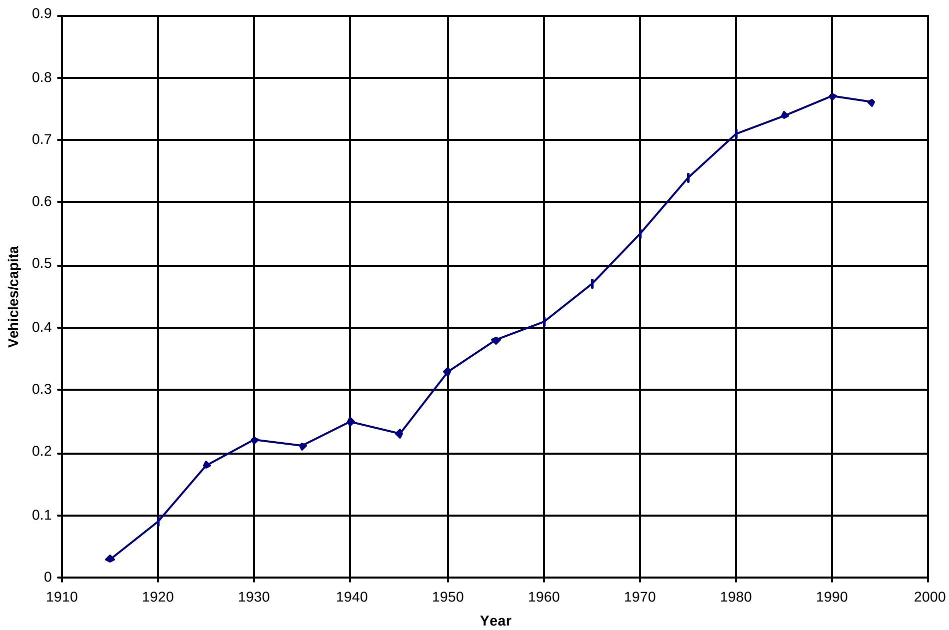


Figure 2-2. Trends in vehicles per capita in the U.S.

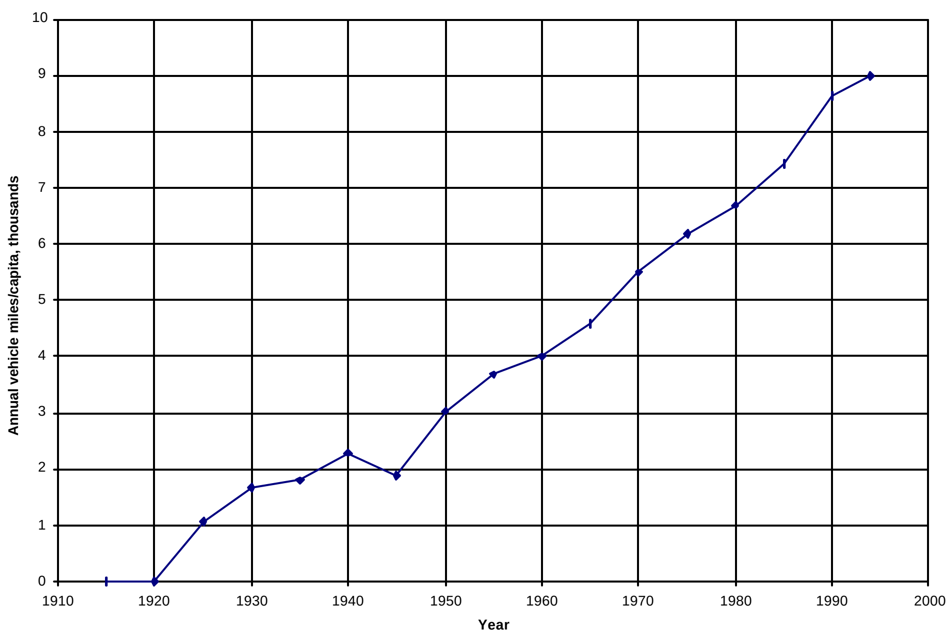


Figure 2-3. Trends in vehicle miles per capita in the U.S.

and gas taxes cover only about 9 to 18% of the cost of transportation (Kunstler 1996). Goldstein (1997) estimates that 25% or more of newly-developed land is committed to roads, parking, driveways, and garages.

The preceding discussion indicates the dominant impact of the automobile on contemporary urban settlements. In order to accommodate more cars and higher rates of utilization, the sizes and proportion of property devoted to vehicles has increased dramatically. One example is the shift from one to two and even three car garages. Parking and other support services have similarly expanded. A key question for the future is whether these trends will continue. If they do, then wet-weather problems will continue to grow in relative importance as will air pollution and noise problems.

Impact of Subdivision Regulations

Southworth and Ben-Joseph (1995) present an overview of suburbia evolution since 1820. They trace the evolution of the current design standards for suburbia, with particular emphasis on city streets. They bemoan the consequences of current standard practices stating (Southworth and Ben-Joseph 1995):

> ***Attempts to reshape the form of the American city are often thwarted by the standards and procedures that have become embedded in planning and development. Particularly troublesome are standards for streets that virtually dictate a dispersed, disconnected community pattern providing automobile access at the expense of other modes. The rigid framework of current street standards has resulted in uniform, unresponsive suburban environments.***

The current residential street design standards which are accepted virtually throughout the United States necessitate a large amount of impervious area per family which consists of wide streets, sidewalks, and driveways.

Contemporary Neighborhoods and Urban Sprawl

Urban areas in the United States are using land four to eight times faster than the growth in population. The New York metropolitan area's population increase over the past 25 years has been only 5%, but the developed land has increased by 61%, replacing nearly 25% of the region's forests and farmlands (Peirce 1994). Cities are spreading over the natural landscapes far faster than population increases or economic progress requires, while older urban districts with their valuable infrastructures are under used or abandoned (Barnett 1993).

In spite of an aggressive program to control urban sprawl and acquire greenways, Portland, OR has grown by nearly 25% since 1980 while expanding its urban area by only 1%. Without such management strategies, the Chicago area's population has grown only 4% in the past 20 years but expanded its urban land by 35%. Between

1960 and 1990, the population of the Baltimore metropolitan area increased by 33% but the amount of land in the region used for urban purposes grew fivefold-by 170% (Katz 1997).

The subdivision is the basic building block of current land use and each parcel within the subdivision is designed to maximize its own identity and privacy. According to Kunstler (1996), the reigning metaphor for the "good life" in the United States is: "... a modest dwelling all our own, isolated from the problems of other people."

However, these properties tend to be much larger than would be suggested by the word "modest" because they attempt to provide a variety of traditional community functions within their individual boundaries such as parks (front and back yard), parking (garages and driveways), and recreation (swimming pools, play areas). Each of these units exists in isolation.

Zoning laws are the chief public instrument used to separate functions in contemporary urban communities. Building the equivalent of Main Street USA in modern America is virtually impossible. It would violate current zoning law provisions such as setbacks, parking requirements, and mixing of land uses. Each major land use function is separated from the others requiring motorized transportation (typically an automobile) to get from one area to another.

Urban sprawl has been a widely debated topic during the past 25 years as automobile-dominated urban transit has become pervasive. Real Estate Research Corporation (1974) analyzed the costs of sprawl for a variety of land use scenarios ranging from uniform low density development to high density, clustered developments. As part of the large on-going effort to protect Chesapeake Bay, the effect of sprawl on land use has been quantified and its implications discussed. This study defined sprawl as (Chesapeake Bay Foundation 1996):

- the haphazard scattering of homes and businesses across the landscape, beyond already developed areas, far from cities and towns.

- an ineffective use of the land, difficult to service with infrastructure and transportation, requiring extensive use of automobiles, and consuming large land areas (CH2M Hill 1993).

- Residential development at a density of less than three dwelling units per acre (CH2M Hill 1993).

Tetra Tech (1996) defines urban sprawl as:

> ***Current development patterns, where rural land is converted to urban uses more quickly than needed to house new residents and support new businesses, and***

> ***people become more dependent on automobiles. Sprawl defines patterns of urban growth which include large acreage of low-density residential development, rigid separation between residential and commercial uses, minimal support for non-motorized transportation methods, and a lack of integrated transportation and land use planning.***

The National Commission on the Environment (1993) criticizes contemporary urban land use pattern by stating:

> ***Meanwhile, sprawling housing developments, shopping centers, highways, and myriad other developments have proceeded virtually unfettered by any sense of respect for the environment and humankind's relation to it. As a result, pollution from non-point sources continues to grow and is increasingly difficult to control; biological diversity is destroyed as habitats are fragmented and eliminated; sprawl development blighted the landscape and precludes cost-effective and environmentally beneficial means of providing transportation and other services; and inner cities at the core of metropolitan areas increasingly are home to people who have been abandoned as hopeless by the rest of U.S. society.***

The impacts of sprawl in the Chesapeake Bay area include (Chesapeake Bay Foundation 1997):

1. Five to seven times the sediment and phosphorus as a forest.
2. Nearly twice as much sediment and nitrogen as compact development.
3. Each person uses four to five times as much land as 40 years ago.
4. Twice as much road building as compact development.
5. Three to four times as many automobile trips per day.
6. Much more air pollution as compact development.
7. Lower tax revenues than the cost of providing these services.
8. Induced relocation of people from central cities and inner suburbs.

Historical Infrastructure Development Patterns

Early infrastructure systems tended to be smaller in size with customers providing some or all of the necessary services or participating in smaller utilities to provide water supply, wastewater, and stormwater services as separate entities. Early transportation systems were often private toll roads. Citizens also formed cooperatives to share the cost of building and maintaining these roads.

The first major call for governmental participation in road construction came in the late

19th century in response to requests from the bicycle community to provide improved roads. Prior to the automobile, railroads provided much of the transportation infrastructure for trips of any significant distance.

Regionalization of urban wastewater infrastructure began in earnest in the 1960's and early 1970's with the federal government providing large subsidies for construction of new wastewater treatment plants and interceptor sewers. Under this program, the urban areas were required to demonstrate that the proposed system was the most cost-effective. Typically, the preferred solution was to build very large regional systems to serve the entire metropolitan area. From a regulatory viewpoint, the agencies strongly preferred larger regional systems since they were easier to administer as opposed to dealing with numerous individual cities and suburbs. The availability of federal subsidies in the range of 75% of the construction cost had a major influence on the decision that "bigger is better". Analogous central systems emerged in water supply, stormwater, and transportation.

Interceptor Sewers and Urban Sprawl

Binkley et al. (1975) evaluated the effect of federally subsidized construction of large interceptors on urban sprawl. The federal government paid 75% of the initial capital cost of interceptors to provide for the existing and future populations. They felt that this subsidy encouraged overdesigning the interceptor sewers. Excess capacity is paid by existing residents who derive little or even negative benefit from it. One alternative funding option is to subsidize only that portion of the interceptor that serves the existing population. Additional capacity would have to be paid by owners of the benefiting property.

This study analyzed 52 interceptor projects. The following conclusions were reached:

- About one half of the total federal investment benefited future growth, not existing customers.
- The costs of excess capacity averaged $145 per capita and was as high as $658 per capita, measured in 1975 dollars.
- Design project periods with a median of 50 years were used. It would be more efficient to use shorter periods of, say 25 years, to reduce uncertainty and to give the existing communities more control over future growth patterns.

Based on this evaluation, Binkley et al. (1975) make the following recommendations:

1. Provide no federal funds for excess capacity. Future growth should pay its own way. Subsidizing this growth will encourage sprawl. Reevaluate interceptor staging of project design in rapidly growing areas. Using shorter design periods reduces the tendency to subsidize future growth. Excess capacity does impose extra cost, especially if it is not used.
2. Use realistic standards for per capita flows. EPA recommended average sewage flows of 100 to 125 gpcd when actual flows average 40-60 gpcd.

3. Improve population forecasting techniques
4. Require consideration of environmental effects of interceptor-induced land use. Increase public participation in the project so that existing stakeholders better understand the environmental and financial implications of the projected project.

Federal Housing and Urban Development Programs

Federal government policies to promote urban economic development have evolved over the past 50 years. Following World War II, urban renewal programs aimed at building affordable housing flourished. The Clinton administration relies on the establishment of empowerment zones and enterprise communities (Moss 1997). These programs have focused on the bricks and mortar aspects of the problem. The Clinton administration's empowerment zone is modeled on the "enterprise zone" concept used in Britain where public investment is attracted by eliminating government regulations and taxes in the worst areas of the city (Moss 1997). According to Moss (1997), the migration of population from the cities to the suburbs is the result of numerous forces including racial and ethnic bias, the construction of high-speed expressways, crime, the decline of urban public schools, and the cultural appeal of low density, single-family housing.

Engel et al. (1996) discuss how the U.S. Dept. of Housing and Urban Development (HUD) and EPA are changing to better integrate their respective missions. They trace the origins of the environmental movement in the United States to late-19th century concerns about poor public health and sanitation conditions in cities and to the need to protect open space and wildlife in undeveloped areas. Early public interventions in housing were brought about by public health concerns about overcrowding, open spaces and urban parks, light and air, sanitary facilities, potable water, and housing and building codes (Engel et al. 1996). The Housing and Urban Development Act of 1968 was intended to have HUD take the lead in implementing a comprehensive urban strategy. The implementation of this act emphasized construction of housing.

Concurrently, major environmental initiatives came on line as a result of numerous legislative mandates. Interestingly, there was little interaction between housing and urban policy advocates and environmental organizations during the 1970s and 1980s and the two programs developed separately. In 1993, the New York Citizens Housing and Planning Council held a conference on housing and environment. Critics argued that environmental regulations were "...endangering the economic viability of the existing housing stock and the rehabilitation or new construction of low-and moderate-income housing." (Engel et al. 1996). This initial effort stimulated other workshops and the development of joint activities between EPA and HUD in areas of common interest such as brownfields.

Engel et al. (1996) synthesize the current situation into four categories arranged in ascending order of difficulty:

1. Procedural reforms: Concern exists that existing environmental regulations, particularly federal mandates, are unduly restrictive and cumbersome. They need to be made more flexible and better integrated into the local planning and permitting process.

2. Balancing of social goals: A natural tension exists between developers and regulators. Strong federal environmental regulation is intended to provide a check against too much control by local development interests. However, these regulations and associated liability have strongly discouraged redevelopment of older sections of urban areas by encouraging builders to go to new areas where environmental cleanup is not an issue. Unfortunately, this contributes to urban sprawl.

3. Urban risk analysis: The comparative risks of environmental stressors need to be prioritized based on the cost effectiveness of reducing these risks. Progress is being made in this area in that individual risk assessments are being done, such as use-based cleanup standards for brownfields. However, it is still difficult for local authorities to develop their own priorities on relative risks because environmental regulations are organized by individual media and pollutants. Trade-offs may not be permitted.

4. Allocation of costs: The issue of who pays for environmental cleanup is at the heart of current debates. During the 1970's, the federal government paid a large share of these control costs. However, this is no longer the case. As of 1990, the federal government was only paying about 30% of pollution control costs (Engel et al. 1996). A significant part of the residual cost falls on local residents, many of whom have limited ability to pay.

Federal Transportation Programs

The federal government has provided the bulk of the financing for the interstate systems that has had a major impact on urbanization since the late 1950's. This support has continued and has been a major inducement for promoting automobile use in urban areas (Littman 1998).

Summary of the Impacts of Federal Urban Programs

Beginning in the 1930's, federal programs to insure mortgages, and associated guidelines for "good" subdivision design, have resulted in widespread adoption of zoning and land use ordinances that foster lower density suburban development. Transportation agencies at all levels have promoted automobile use by providing large subsidies for this mode of transportation and mandating "free parking" and generous widths on little used streets. USEPA construction grants for wastewater treatment during the 1970's encouraged construction of large interceptor sewers and centralized wastewater treatment plants. The large amount of "excess capacity" in these systems encouraged low density development as cities sought customers to utilize this available capacity. Liability concerns with renovating brownfields in urban areas encouraged migration away from the core city to greenfield areas. Recent years have seen a rekindling of interest among federal agencies to look at urban systems in a more unified manner in order to promote more sustainable communities.

Possible New Approaches

Neo-traditional Neighborhoods

One attempt to develop modified urban land use patterns is called the New Urbanism school. New urbanism is also called neo-traditional planning, traditional neighborhood development, low density urbanism, or transit oriented development (Kunstler 1996). The key component of the "new approach" is to return to the pre World War II practice of designing urban neighborhoods with a mix of land uses rather than segregating land uses by function as currently exists. Features of traditional neighborhood developments (TND) include the following (Chellman 1997):

1. Mixed land uses.
2. Gridiron street pattern to maximize circulation. The goal is to maximize connectivity of streets, not the opposite.
3. Most TND streets are designed to minimize through traffic by using tee intersections.
4. Alleys.
5. Garages in rear of house facing alley.
6. Smaller front yard with porches to reflect the increased friendliness of neighborhood.
7. Higher densities that promote alternative forms of transportation to the automobile. Typical TND densities in the United States are 6-10 dwelling units per acre.
8. Designed to maximize non-motorist mobility for residents and visitors.
9. Residential streets are designed for shared use; they are not designed merely to optimize automobile movement. Examples of narrower streets in traditional neighborhoods include (Chellman 1997, p. 25) two lane-two parking lane streets with a 25 foot curb to curb dimension (Seattle, WA), 28 to 32 feet wide (Georgetown in Washington, D.C.), 21 feet wide (San Francisco, CA), 22 feet wide (Madison, WI), 26 to 30 feet wide (Portsmouth, NH), and 18 to 28 feet wide (Portland, OR). As Chellman (1997) points out, the narrower streets reduce traffic speeds to 10-20 mph, thus improving safety for other users.
10. The scale of the design is based on the primary user being a pedestrian, not an automobile driver. For example, signs are smaller.
11. TNDs are sized based on walkability. Thus, they range in size from 40 to 125 acres.
12. Most commercial units have residences located on upper floors of the TND project.
13. On-street parking is allowed.

A prominent example of a neo-traditional community is Celebration, a new development by Disney Corporation near Orlando, FL. This 4,900 acre development will house 20,000 residents in a mix of land uses. These new communities try to reduce the impact of the automobile on urban settlements. Smaller streets are used in the neighborhoods. Alleys with garages are used so that streets will be lined with front porches and lawns, not garage doors and driveways. Open space including pocket parks are an integral component of these new communities. Ben-Joseph (1995) presents several examples of such developments in the Netherlands, Germany, England, Australia, Japan, and Israel. Another example in the U.S. is Seaside, FL (Mohney and Easterling, eds. 1991).

The preceding examples of "new urbanism" reflect current attempts to convince Americans that alternative options exist. However, many long-term examples already exist in older cities of the United States and Europe.

Newsweek (1995), in an article based on interviews with leading New Urbanism proponents, Andres Duany, Elizabeth Plater-Zyberk, Peter Calthorpe, and Henry Turley, summarizes 15 basic tenets of the new urbanism:

1. Give up big lawns: they increase sprawl, require large amounts of irrigation water, and increase alienation.
2. Bring back the corner store: a simple development that both brings local residents together and a convenience that does not require a 10-mile trip to the supermarket.
3. Make the streets skinny: plan neighborhood streets for walking not driving.
4. Drop the cul-de-sac: although a "dead-end" neighborhood prevents through traffic, it chokes that one road that connects the neighborhood with the rest of the world.
5. Draw boundaries: limit the city's physical size; don't let population increase cause sprawl.
6. Hide the garage: neighborhoods are for living, not parking.
7. Mix housing types: avoid monoculture neighborhoods and invite diversity through development.
8. Plant trees curbside: beautify the places we travel and walk.
9. Put a new life into old malls: plan shopping centers not entirely around the consumer, but strive to bring together a community.
10. Plan for mass transit: encourage alternatives to the automobile.
11. Link work to home: break the idea that one has to travel a great distance to work.
12. Make a town center: focus a development around a public center
13. Shrink parking lots: business can share parking.
14. Turn down the lights: light streets for the pedestrian, not the automobile.
15. Think green: instead of endless manicured green carpets, invite nature into the community.

The wave of interest in New Urbanism concepts of urban planning has rekindled the debate regarding the pros and cons of traditional neighborhood developments. Chellman (1997) presents an overview of the debate and evaluates the transportation aspects of traditional neighborhood development. Ewing (1996) evaluates new urban developments and compares them to traditional developments. He presents a list of best development practices for land use, transportation, housing, and environmental practices. No work was found that evaluated the impact of neo-traditional development on urban water infrastructure. Accordingly, a preliminary evaluation of this topic is presented in this report.

Related EPA Activities Dealing with Urban Growth Patterns

In addition to the activities of the National Risk Management Research Laboratory that is sponsoring this study, other groups within US EPA are interested in issues of urban development and its environmental impacts. These groups are discussed here.

Green Development

U.S. EPA's Office of Wetlands, Oceans and Watersheds is developing the Green Development approach to make urban growth and development work with existing environmental resources. Tetra Tech (1996) compiled a list of case studies of innovative urban development. The case studies are divided into the following categories:

- Urbanizing suburbs and areas where infill has successfully occurred (See Table 2-2).

- Intermodal transport policies that consider environmental impact (See Table 2-3).

EPA's air quality control program is encouraging methods to reduce the demand for vehicle travel by a variety of means including charging systems (ICF Incorporated and Apogee Research Inc.1997).

Green development achieves its goals using the following (Tetra Tech 1996):

1. Flexible zoning and subdivision regulations.
2. Management of growth through agriculture and natural resources preservation.
3. Comprehensive and integrated site planning.
4. Reduction in site imperviousness.
5. Restoration of the site hydrologic regime to mimic the natural or predevelopment condition.
6. Maintenance of surface water and groundwater quality and minimization of the generation and off-site transport of pollutants.
7. Minimization of disturbance of riparian habitat functions.
8. Preservation of terrestrial habitat ecological functions and maximizing conservation of woodland and vegetative cover.
9. Use of compact, pedestrian-friendly development practices.

Studies of Chesapeake Bay

The Chesapeake Bay Foundation (1996) advocates the following principles to avoid sprawl:

1. Channel development into “growth areas,” that is, compact mixed-use patterns in and adjacent to existing cities and towns.
2. Create “growth boundaries” to keep sprawl out of open lands where farming, forestry and recreational activities should prevail.
3. Maintain existing highways, improve local roads, and use transit to connect and organize land uses in growth areas.
4. Revitalize existing towns and cities.

Table 2-2. Case studies on "urbanizing" suburbs and areas where infill has successfully occurred (Tetra Tech 1996).

Case Study Name	Location	Economic Analysis Included?
California Infill Development Program	California	No
Downtown Master Plan*	City of West Palm Beach, FL	No
Florida Main Street Program	State of Florida	No
Grand Central Square	Los Angeles, CA	Yes
Memorial Park	Richmond, CA	Yes
Mizner Park	Boca Raton, FL	Yes
River Place	Portland, OR	Yes
Uptown District	San Diego, Ca	Yes
Ballston	Arlington, VA	No
Main Street	Huntington Beach, CA	No
Downtown Redlands	Redlands, CA	No
Whittier Boulevard	East Los Angeles, CA	No
The Eastward Ho! Initiative	South Florida	No
Fearrington	Near Chapel Hill, NC	No
Fairview Village	Near Portland, OR	No
Downtown area	Mashpee, MI	No
Downtown area	Boca Raton, FL	No
Revitalization Plan	Orlando, FL	No
The Florida Avenue Project	Miami, FL	No
The Jordan Tract	Mount Pleasant, SC	No
North Boulder	Boulder, CO	No
South Martin County	Martin County, FL	No
Master Plan	Port Royal, SC	No
Montgomery Village	Montgomery Township, NJ	No
Lake Park Village	Union County, NC	No
Oak Ridges Moraine	Toronto, Canada	No
Peaks Branch	Dallas, TX	No
Dorsey Woods	Arlington, VA	Yes

Brownfield Redevelopment

The US EPA is promoting the redevelopment of brownfields in older urban areas. A review of this program highlights many of the challenges of reversing the trend from continued development of green fields on the periphery of urban areas to redevelopment of existing areas. Challenges include technical, socio-economic, and liability issues as discussed below. Barnette (1995) lists three advantages of redeveloping brownfields:

- Brownfields are properly zoned and thus well suited for industrial and commercial use.
- The civil infrastructure and utilities necessary for industrial operations are already in place at many brownfield sites.
- Brownfield redevelopment preserves the nation's virgin land and natural resources.

Table 2-3. Case studies using intermodal transportation policies that consider environmental impacts (Tetra Tech 1996).

Case Study Name	Location	Modes Provided ($)[1]
Effects of Interstate 95 on Breeding Birds	Maine	A
For Animals. It's the Road to Safety	Washington, DC	A
Haymount	Caroline, Co., VA	T,A
Skinny-Streets & One-sided Sidewalks: A Strategy…Paradise	Olympia, WA	A
I-287 it and They Will Drive On It	Wanaque, NJ	A($)
For Many, Gas Guzzler is Necessary Tool, Not a Toy	Cllifton Park, NY	A($)
The Road Less Noisy: How America is Muffling the Highways	Colorado	A($)
Portland's Pedestrian Master Plan	Portland, OR	P
City of Toronto	Toronto, Canada	T,A
City of Seattle Bicycle Program	Seattle, WA	B
State of Washington Transportation Planning	Washington	T,A,P
Core Area Requirements to Support Non-Auto Trips, New Jersey Transit	New Jersey	T,A,P
Designing for Transit, Integrating Public Transportation and Land Development	San Diego Metropolitan Area	T,A,P
Guide to Land Use and Public Transportation	Snohomish County, WA	T
The Citizen Transportation Plan for Northeastern Illinois	Chicago Region, IL	T,A($)
Transit-Supportive Land Use Planning Guidelines*	Ontario, Canada	T,A,P
TCEA-Transportation Concurrency Exception Area	Delray Beach, FL	T,A,P,B
Smart Development Program	State of Oregon	T,A($)
The Crossings	Mountain View, CA	T,A,P($)
Old Pasadena	Pasadena, CA	T,A,P
North Thurston UGMA	Thurston County, WA	T,A,P
North Boulder	Boulder, CO	T,A
South Martin County	Martin County, FL	T,A,P
Revegetation along US 189	Provo Canyon, UT	T
Stream Restoration in Boulder	Colorado	P,B
Rail Plan on the Wrong Track	Maryland	T($)
MSHA Grown, Don't Mow Program	Maryland	T

1) A: Auto, B: Bicycle, P: Pedestrian, T: Transit, $: Economic analysis included.

Collatin and Bartsch (1996) discuss three major concerns regarding brownfield redevelopment: the high cost of cleanup, the uncertainty about liability and procedures, and a negative public attitude towards old facilities. Cleanup costs are an upfront cost for developers and include required site environmental assessments for all properties. Given the initial assessment, the developer still faces major uncertainties about the ultimate final cost. Thus, lending institutions are understandably reluctant to become involved in such high-risk ventures. Review procedures are complicated by not having clear guidelines on the required level of control and the extent of the public review process. Lastly, the above concerns and a recent history of negative attitudes towards these properties further reduces their desirability. Amedudzi et al. (1997) provide an overview of brownfield redevelopment issues at the federal, state, and local levels.

The follow existing brownfield demonstration projects are explicitly linked to urban water systems (Colatin and Bartsch 1996):

1. Birmingham, AL: Link environmental protection approaches involving flood control and stormwater/groundwater contamination reduction with remediation of soil and site-specific contamination, and develop consortium of community leaders to direct resources to targeted areas.

2. Erie County, NY: Brownfield cleanup as part of a large waterfront redevelopment project.

3. Laredo, TX: Seek conversion of brownfield into waterfront recreation area near campus of a community college.

4. Lima, OH: Focus on remediating and redeveloping 200-acre industrial park and support ongoing river corridor redevelopment activities in order to enhance water quality and provide greenspace.

5. Pritchard, AL: Remediate extensive organic chemical contamination of city's water supply by using State Enterprise Zone tax credits to encourage investment.

Sustainability Principles for Urban Infrastructure

A general guiding principle for designing innovative urban stormwater management systems for the 21st century is that they promote sustainable development. A popular general definition of sustainable development is:

> ***Development that meets the needs of the present without compromising the ability of future generations to meet their own needs (World Commission on Environment and Development 1987).***

The following principles are suggested for sustainable infrastructure systems for the 21st century:

1. Ideally, individual urban activities should minimize the external inputs to support their activities at the parcel level: For water supply, import only essential water for high valued uses such as drinking water, cooking, showers and baths. Reuse wastewater and stormwater for less important uses such as lawn watering and toilet flushing. Minimize the demand for water by utilizing less water intensive technologies where possible. For transportation, minimize the generation of impervious areas, especially directly connected impervious areas, for providing traffic flow and parking in low use areas.

2. Minimize the external export of residuals from individual parcels and local neighborhoods: For wastewater, export only highly concentrated wastes that need to be treated off-site. Reuse less contaminated wastes such as shower water for lawn watering. For storm water, minimize off-site discharge by encouraging infiltration of less contaminated stormwater and using cisterns or other collection devices to capture and reuse stormwater for lawn watering and toilet flushing.

3. Structure the economic evaluation of infrastructure options to maximize the incentive to manage demand by using commodity use charges instead of fixed charges: For water supply, assess charges based on the cost of service with emphasis on commodity charges. Charges should be a combination of a level of service that specifies flow, quality, and pressure. For wastewater, assess charges based on the cost of service with emphasis on commodity charges. Charges should be a combination of a level of service that specifies flow and quality. For stormwater, assess charges based on the cost of providing stormwater quality control for smaller storms and flood control for larger storms. Charges should be based on the imperviousness with higher charges for directly connected imperviousness and the nature of the use of the impervious areas and their pollutant potential. Some charge should be assessed for pervious areas. Credit should be given for on-site storage and infiltration. For transportation, assess charges for transportation related imperviousness directly to users as fees per mile for travel and fees per hour for parking in order to encourage demand management and switch to more sustainable modes of transportation.

4. Assess new development for the full cost of providing the infrastructure that it demands, not only within the development, but also external support services.

5. Implement policies to make drivers pay the full cost of using personal automobiles.

The following list of other goals provides additional criteria for more sustainable new communities. These topics overlap and can be consolidated down to a much smaller set of principles.

1. Re-develop vacant or low-density development within currently developed areas at higher intensities.
2. Design comprehensive, mixed-use neighborhoods instead of isolated pods, subdivisions and developments. The spaces between neighborhoods should consist of functional open space such as farms, grazing areas, gardens, parks, playgrounds, bikeways, jogging trails and the like.
3. Encourage telecommuting and the infrastructure necessary to make it work.
4. Do a comprehensive accounting of infrastructure costs that reflects social and environmental costs as well as economic costs. Current investments based on partial and incomplete accounting systems are considered to be factors in urban sprawl and the inability of infrastructure capacity to keep pace with these urban development patterns.
5. Develop a community designed for people first, that does not damage the natural environment, that enables a healthy, active lifestyle, where human interaction is an everyday event (Goldstein 1997).
6. Housing, stores, and employment will be accessible (less than 20 minutes) to each other by walking, biking and transit (Goldstein 1997):
7. With regard to environmental impacts, the City of Dreams will have the following benefits (Goldstein 1997):
    a. Reduce energy demand by 75%.
    b. Reduce water use by 65%.
    c. Reduce solid waste by 90%.
    d. Reduce air pollution by 40%.

Much general information on this subject is available on the internet, (e.g., see $mart Growth Network-www.smartgrowth.org).

Sustainability and Optimal Size of Infrastructure Systems

While the notion that "bigger is better" still persists, some argue that these systems are not sustainable. Problems with larger systems include:

1. Large organizations are necessary to manage these systems.
2. Large organizations with monopoly powers tend to be inefficient and less responsive to changing needs.
3. Complex cost sharing arrangements need to be developed to fairly charge each group for its share of the cost of the system.
4. Complex political institutions are needed to govern these systems that cross city, county, and even state boundaries.
5. Part of the savings associated with regional systems results from transferring problems from area to area so as to take better advantage of the assimilative capacity of the receiving environment. While such solutions may reduce

costs overall, they may be highly objectionable to citizens in those parts of the service area that receive a disproportionate share of the negative effects of such transfers, (e.g., added flood hazard , traffic noise, more polluted water).

6. Large regional systems are inefficient if recycling of treated wastewater and stormwater is desired since it is necessary to pipe and pump this water back through the entire system.
7. The failure of larger systems causes more serious consequences since larger areas are affected and illicit discharges are concentrated at fewer points.
8. Customers are less aware of the nature of the problems that they cause and are therefore less receptive to their responsibility to better manage their demand for the service.
9. The strong tendency for urban sprawl that has accompanied the creation of these regional systems makes them even less efficient due to the added distribution costs associated with more dispersed development.
10. It is necessary to build large amounts of excess capacity into these regional systems. Thus, the existing customers pay this added cost. The primary beneficiaries of this largesse are new customers. Correspondingly, the governing agency has a strong incentive to promote the growth of the area to help pay for this unused capacity.
11. Regional systems serve a heterogeneous group of customers including domestic, commercial, and industrial users. Thus, the nature of the wastes are harder to predict and the design must be upgraded accordingly. The use of a regional system encourages off-site discharge of wastes instead of prevention or treatment at the source.
12. Once established, it is difficult to restructure large organizations who enjoy monopoly power to provide the infrastructure service.

Given the above concerns, one of the main themes of this report is the need to rethink this basic “bigger is better” premise that has guided water infrastructure development during the past 30 years. Perhaps, bigger is not better.

Models for Evaluating Future Infrastructure

Beginning in the 1960’s, large-scale efforts were made to develop urban planning models that link land use, transportation, and infrastructure including environmental impacts. Large simulation models were developed to support these efforts. These models included the critical interaction between provision of infrastructure and land use. This is particularly important in showing the impact of transportation on land use. These early models were severely limited due to use of relatively primitive computers, lack of good databases, and poor knowledge of the underlying cause-effect linkages of urban dynamics (Lee 1973). Few urban models were developed after the early 1970's but a renaissance in the development and use of these models began to occur in the early 1990s (Wegener 1994). The resurgence of interest in urban planning models in the 1990's is partially due to the renewed recognition of the need to link transportation-land use models to urban environmental systems models.

Integrated urban models to evaluate the overall efficacy of alternative growth scenarios do not exist. However, there are individual models for water, wastewater, stormwater, and transportation. These models need to be integrated with each other and with land use models at both the micro (neighborhood) scale as well as the macro (urban area) scale. Preliminary evaluations using simple models are presented in this report.

Research Initiatives Related to Urban Infrastructure
Until recently, research support has been unavailable for evaluating alternative infrastructure systems. However, the National Science Foundation has initiated research programs in this area. Zimmerman and Sparrow (1997) summarize the results of an NSF sponsored workshop on integrated research for civil infrastructure. This is the third workshop on this subject since 1993. The participants strongly recommended a holistic view of infrastructure development. Sustainable infrastructure is defined as: "Achieving a balance of human activity (including human settlements and population growth) with its surroundings, so as not to exceed available resources."

Infrastructure sustainability is discussed around four topics:

1. Life-cycle engineering (LCE), that is, a process that incorporates into design the "true costs" of construction, operation, maintenance, renewal, and any other requirements over the expected lifetime of the facility. LCE includes design, construction, and repair, rehabilitation, reconstruction, retirement, and removal. Current costing methods and related institutions hamper LCE in the following ways:
    a. Incentives and statutory restrictions often favor "least-first-cost" contracting.
    b. New capital projects are often favored politically over maintenance or rebuild contracting.
    c. Tight budgets preclude field inspections, favor corrective over preventive maintenance, and encourage the use of minimal specifications for materials and structures.

2. Technology investment
    a. Mechanisms are needed to integrate infrastructure design, construction and maintenance. For example, integrated utility corridors provide a way to reduce the life cycle cost of infrastructure, particularly maintenance of subsurface infrastructure.
    b. Innovative approaches for technology investment at every point in the life cycle of infrastructure systems, (e.g. develop more durable materials, better monitoring and diagnostic techniques, better designs, and more rational methods for determining design safety factors throughout the lifetime of the infrastructure).

3. Performance measures
    a. Research is needed on the appropriate adaptation of process control management procedures in conjunction with advanced probabilistic and reliability methods for urban infrastructure systems.
    b. Research is needed on proper output performance measures for infrastructure and how it relates to costs.
    c. Performance measures need to be supported by direct monitoring of the physical state of the system and changing public expectations for use, capacity, and performance.

4. Project management
    a. A new generation of simulation and optimization models are needed to address both the new "intelligent" infrastructure, new model characteristics, and new cultures of the consumers.
    b. Encourage Design-Build-Operate (DBO) contracting mechanisms that will promote the evaluation of projects on a life-cycle basis. At present, using least cost criteria for design and construction leads to much higher maintenance costs over the life of the project. If the designer and builder also has to operate the infrastructure, they will have the proper incentives to minimize the entire life cycle cost, and not just the initial cost. Such procedures are already being used in Europe and Japan.

Transportation/Land Use Strategies to Alleviate Congestion
Congestion in urban transportation systems can be alleviated by expanding the capacity of the existing system. The capacity of the existing system can be expanded by improved traffic engineering and rescheduling work hours. also, demand can be managed by providing added incentives to use alternative modes of transportation, managing parking availability, promoting more transportation efficient land use patterns, and/or encouraging trip reduction through telecommuting or work at home options (Deakin,1995).

Projected Future Trends
Projected general trends are:

1. Continuing migration of population to cities throughout the world. By the year 2000, more than half of the world's population will live in cities. These cities will continue to grow in size with numerous mega-cities developing throughout the world. Okun (1991) summarizes the migration of people to urban areas around the world. In 1950, less than 30% of the world's population lived in cities. This percentage will exceed 50% by the year 2000. In developed countries such as the U.S., over 75% of the people live in cities.
2. The spatial settlement patterns of future urban development may differ significantly from current patterns. Population is being redistributed away from the core of the cities. Modern telecommunications could have a

profound impact on settlement patterns and transportation needs.

3. Public expectations about levels and types of service are continually changing as standards of living and life styles change.
4. The magnitude and distribution of investments in infrastructure are changing. Government subsidies of infrastructure are decreasing in some areas, (e.g., wastewater treatment plants), and increasing in other areas, (e.g., major highways and interstate expressways). The timing and lengths of budgetary cycles are changing with efforts to better integrate life cycle costs into new design and construction.

Origins of Stormwater in Urban Areas

Introduction

The purpose of this section is to evaluate the nature of the quantity of stormwater runoff in urban areas and to evaluate the relative importance of various sources. Water quality impacts are evaluated in Chapter 5.

Stormwater falls onto pervious or impervious areas. Runoff occurs after the infiltration capacity has been exceeded. Impervious areas have a very small amount of initial storage capacity whereas pervious areas have much larger initial storage capacities depending on the soil type and antecedent conditions.

A primary goal of sustainable water infrastructure systems is to maximize the management of the problem at the source, that is, the parcel or local level. Thus, it is important to understand the movement of water at this scale. An evaluation of the nature of the rainfall-runoff relationship at the neighborhood level is presented in the next section. Then, detailed discussions of the nature of impervious and pervious areas are presented in the later sections.

Rainfall-Runoff Relationships at the Neighborhood Scale

An integrated urban stormwater management program should provide a sustainable solution to the problem of handling storms of all sizes from micro-storms to major floods. Early studies in Chicago showed that most of the annual volume of runoff is associated with smaller storms as indicated in Table 2-4 (APWA 1968). For this Chicago catchment, 10.8 inches of runoff resulted from 34.7 inches of precipitation that occurred during 122 events. About 50% of the runoff resulted from precipitation of 0.5 inches or less, that roughly corresponds to storms that occur, once a month, on the average. Nearly 75% of the runoff volume is from storms that result from precipitation of one inch or less. Thus, the key point is that these smaller storms account for the majority of the runoff volume. Similar results were reported later by Heaney et al. (1977) and Roesner et al. (1991).

Early studies in Chicago by Harza Engineering and Bauer Engineering (1966) demonstrated that runoff is a nonlinear function of precipitation as shown in Figure 2-4. Up to rainfalls of two inches with corresponding runoff of about 0.6 inches, the relationship is linear with contributions only from the impervious areas, approximately an equal mix of runoff from directly connected roofs and streets and alleys. For rainfalls greater than two inches, runoff from pervious areas begins and becomes the major source for rainfalls greater than four inches.

Pitt and Voorhees (1994) show the nature of runoff for a residential area in Milwaukee as shown in Figure 2-5. For this case study, all of the runoff came from streets, driveways, and roofs up to precipitation depths of 0.1 inches. In this range, about 80% of the runoff came from transportation related imperviousness. As the rainfall depths increase, the landscaped areas become more significant sources of total runoff. At the one inch depth, landscaped areas contribute about 40% of the runoff.

These relative contributions are site specific but it is safe to conclude that the initial runoff is the runoff from the directly connected impervious areas. Impervious area (IA) is defined as land area that infiltrates less than 2% of precipitation that falls onto its surface directly or runs onto this surface. Directly connected impervious area (DCIA) is the IA that drains directly to the storm drainage system.

Table 2-4: Types of storms contributing to stormwater runoff in Chicago,IL (APWA 1968).

Precipitation (inches)	Average Runoff (inches)	Events per year	Precipitation (inches/yr.)	Runoff (inches/yr.)	% of Runoff	Cumulative % of Runoff
0.1	0.03	78.00	7.80	2.34	21.6	21.6
0.3	0.09	19.80	5.94	1.78	16.4	38.0
0.5	0.15	9.60	4.80	1.44	13.3	51.3
0.7	0.21	5.20	3.64	1.09	10.1	61.4
0.9	0.28	3.20	2.88	0.90	8.3	69.7
1.1	0.35	2.40	2.64	0.84	7.8	77.4
1.3	0.42	1.30	1.69	0.55	5.0	82.4
1.5	0.49	0.92	1.38	0.45	4.2	86.6
1.7	0.56	0.53	0.90	0.30	2.7	89.3
1.9	0.63	0.36	0.68	0.23	2.1	91.4
2.1	0.7	0.22	0.46	0.15	1.4	92.9
2.3	0.76	0.14	0.32	0.11	1.0	93.8
3.0	1.26	0.53	1.59	0.67	6.2	100.0
Total		122.20	34.73	10.84		

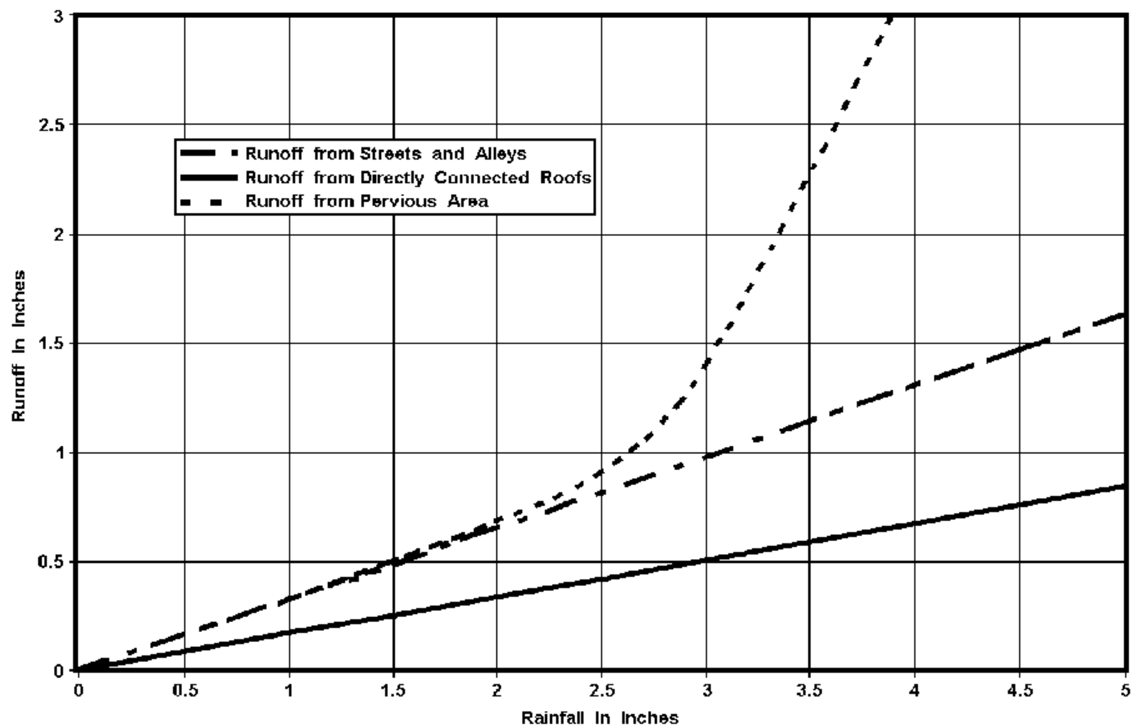


Figure 2-4. Rainfall-runoff relationships for unit area, Chicago,IL (Harza and Bauer, 1966).

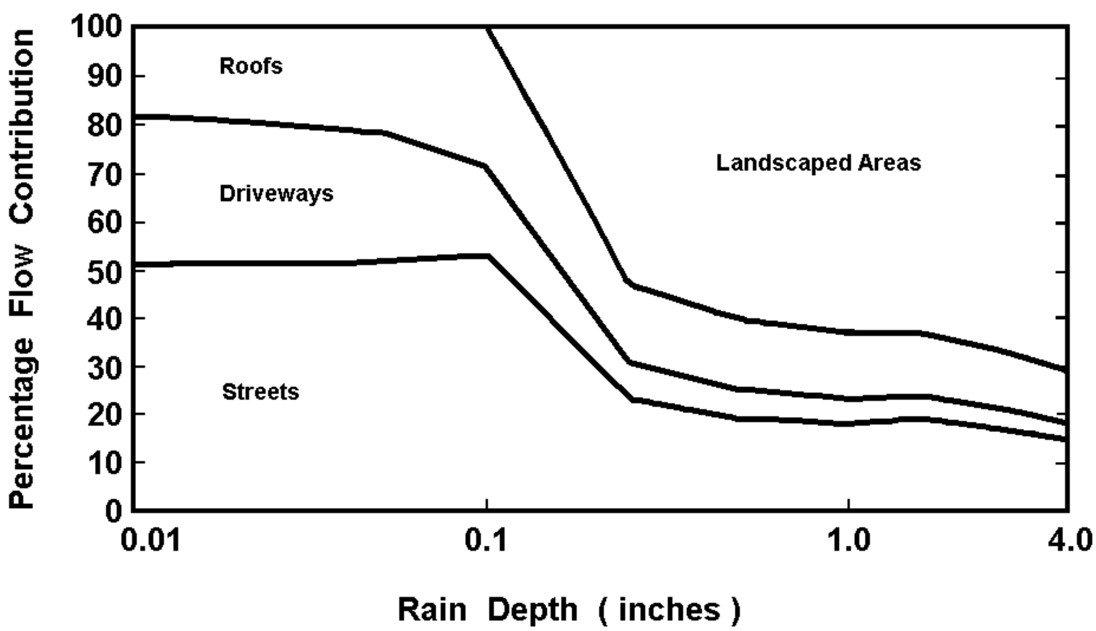


Figure 2-5. Flow sources for example medium density residential areas having clayey soils, Milwaukee, WI (Pitt and Voorhees, 1994).

Imperviousness has been suggested as a good single indicator of the extent of urbanization as far as stormwater impacts are concerned (WEF-ASCE 1998). For example, Schueler (1994) shows the dependence of the runoff coefficient on imperviousness. This relationship is based on evaluation of more than 40 runoff monitoring sites as part of the Nationwide Urban Runoff Program (NURP) studies. While a generally positive trend is evident in Figure 2-7, a large variability remains indicating that imperviousness alone is not an adequate predictor of runoff.

Population density has been used to predict imperviousness as shown in Figure 2-8 (Heaney et al. 1977). A primary unresolved source of variability in these results is the use of different bases for defining the service area. Some of these studies used small areas on the scale of blocks while others used aggregate data for much larger areas that included other land uses such as schools, parks, and commercial areas. Thus, the results vary widely.

Previous Studies of Imperviousness

Schueler (1996) cites the results of a recent study by the city of Olympia, WA which shows the components of imperviousness for a variety of land uses as shown in Table 2-5. Road related imperviousness is seen to comprise 63% to 70% of the total. Schueler (1995) contends that cluster development can reduce the imperviousness by 10-50% depending on the lot size and road network. Arnold and Gibbons (1996) show an example of the effect of cluster development in reducing imperviousness from 17.5% to 10.7%. Schueler (1995) presents a detailed analysis of the relationship between land use and imperviousness. He discusses alternative street designs, parking provisions, expected imperviousness, pollutant loads, and BMP options for control.

Debo and Reese (1995) show how to adjust SCS curve numbers based on the proportion of imperviousness that is directly connected. Unit pollutant loadings are often expressed in terms of curb lengths. Novotny and Olem (1994) show a relationship between percent imperviousness and curb length per unit area. The American Public Works Association (1968) estimated curb length as a function of population density. The use of population density as the independent variable is subject to significant error because it can be defined in several ways. The density varies significantly depending upon whether open space or other land uses such as streets are included in the area.

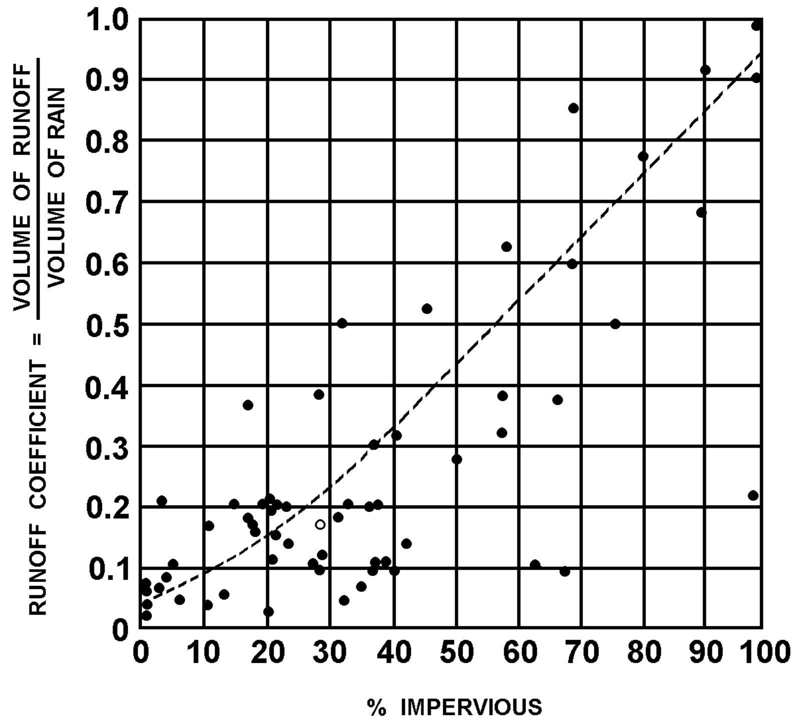


Figure 2-6. Relation of the coefficient of runoff for urban areas to imperviousness (Schueler 1994).

Table 2-5. Site coverage for three land uses in Olympia, WA (Schueler 1996).

	Average Approximate Site Coverage, %		
Surface Coverage Type	**High Density Residential (3-7 units/acre)**	**Multifamily (7-30 units/acre)**	**Commercial**
1. Streets	16	11	3
2. Sidewalks	3	5	4
3. Parking/driveways	6	15	53
4. Roofs	15	17	26
5. Lawns/landscaping	54	19	13
6. Open space	n/a	34	n/a
Total impervious surface (1-4)	40	48	60
Road-related impervious surface (1-3)	25	31	86
(Road-related as a percentage of total impervious coverage)	(63%)	(65%)	(70%)

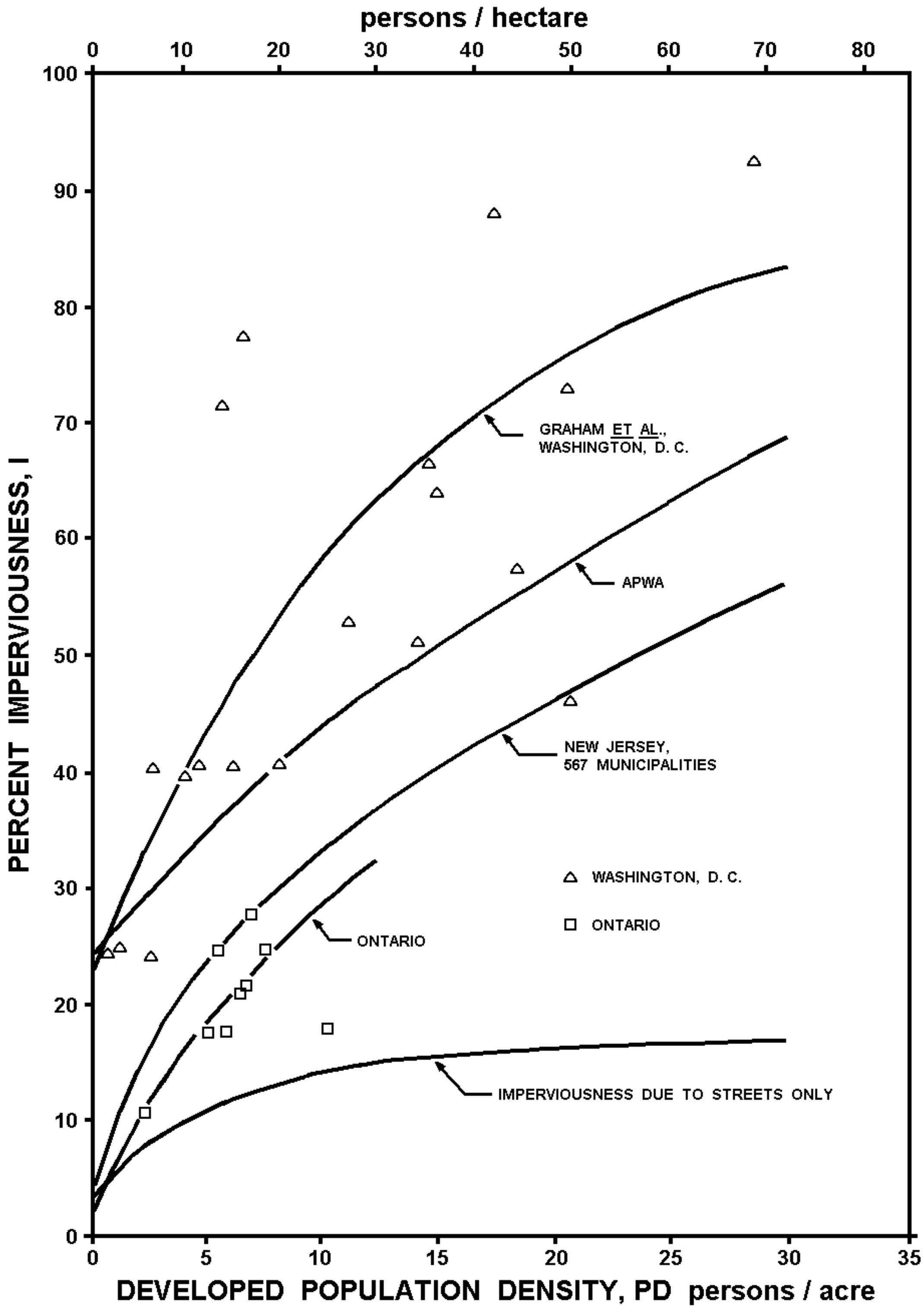


Figure 2-7. Imperviousness as a function of developed population density (Heaney et al. 1977).

Sources of Urban Runoff
A sketch of a contemporary residential lot and associated right of way (ROW) is shown in Figure 2-8. Each parcel consists of the development on the lot itself plus the adjacent development in the right of way that provides infrastructure services for this parcel, plus services for adjacent parcels. For this illustration, the overall area of the lot plus the ROW is summarized below:

Overall lot plus ROW area, sq. ft. =	7,020
Lot area, sq. ft. =	4,980
ROW area, sq. ft. =	2,040

For this case, about 71% of the total area is devoted to the lot and the ROW occupies the remaining 29%. This is close to a rule of thumb that says that the ROW occupies about 25% of the developable land area. When calculating development densities, it is important to define whether the denominator is the lot area only, the lot plus ROW area, or lot plus ROW plus other land uses including open space.

The percent imperviousness for the lot and ROW is 50.4% while it is only 38.2% for the lot only. The most dramatic statistic is the breakdown of imperviousness by function. Only 34% of the imperviousness is due to the living area itself. Nearly 60% of the imperviousness is due to providing for vehicles. The remaining 7% of the imperviousness is due to sidewalks.

The directly connected imperviousness (DCIA) is the most important component as far as causing stormwater runoff quantity and quality problems. About 80% of the DCIA is due to vehicle related imperviousness, predominantly the street and the portion of the driveway that drains to the street. While this percentage will vary, this illustration does indicate the dominance of vehicle related DCIA in contemporary urban development. It is now standard practice to discharge roof runoff onto pervious areas, particularly in lower density developments with well drained soils. Thus, rooftops are no longer the predominant source of DCIA; rather streets and driveways have grown in relative importance as the number of vehicles has increased. It is instructive to examine a cross section of residential land use to generate a database from which more general inferences can be made regarding how imperviousness is affected by land use.

Categories of Urban Catchments

A popular way to classify urban land uses is to define various categories of residential land use, (e.g., low density, commercial, industrial, and public land uses). Associated with each land use is an estimated imperviousness. A limitation of such general measures is that they don’t provide a breakdown on the nature of the imperviousness. Another limitation is lack of specificity in how the area is defined as discussed above. A

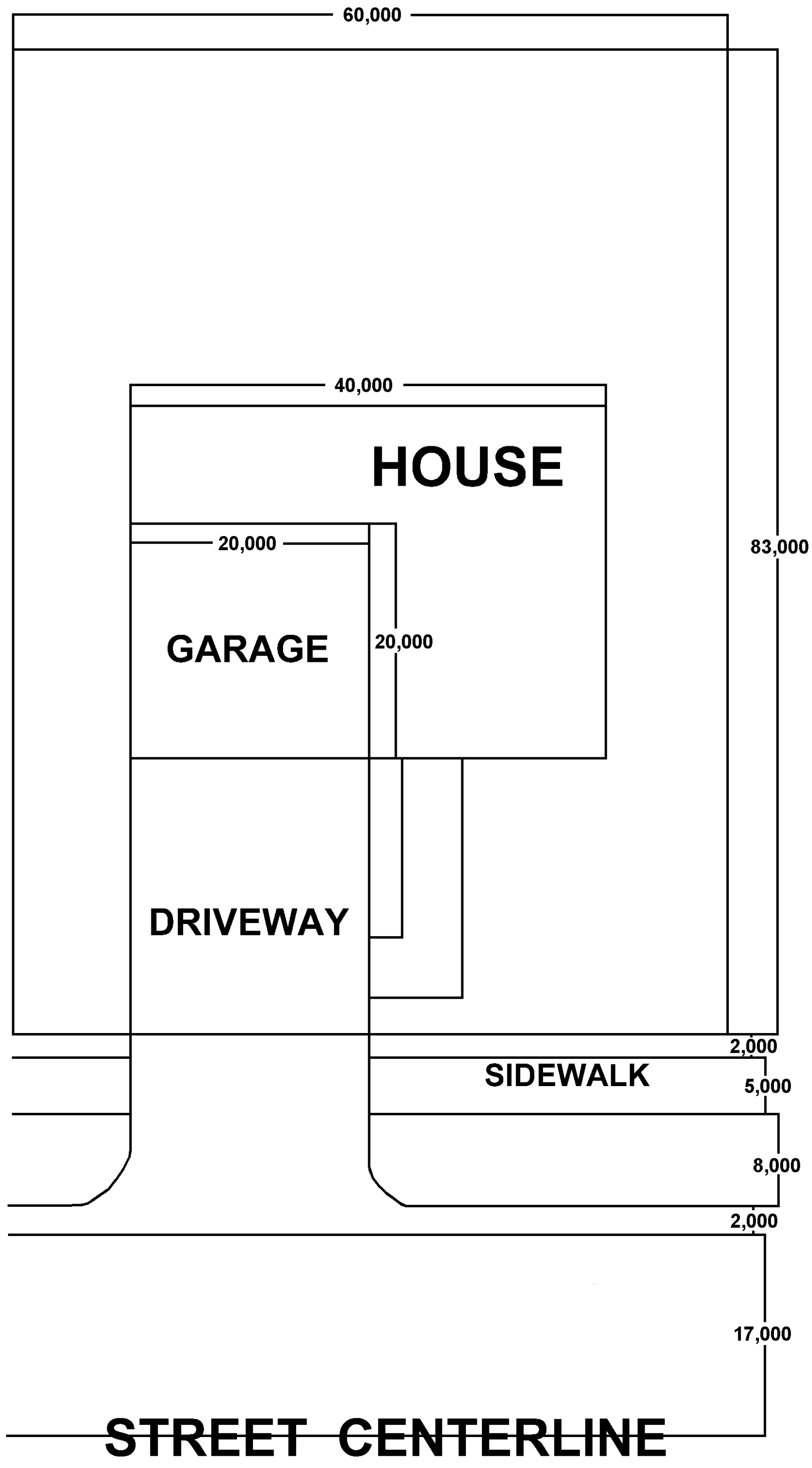


Figure 2-8. Example urban lot.

more functional way to partition urban areas is by the nature of the imperviousness and whether it is directly connected to the storm drainage system. For residential areas, the total land area can be divided into two major components: residential lots, and right-of-way, as shown in Figure 2-8. The lot portion of the area is divided into the following components:

1. House
2. Garage
3. Part of driveway
4. Yard
5. Walkway to dwelling unit
6. Pool
7. Deck/shed

The ROW portion of the area is divided into the following components:

1. One half of street consisting of driving and parking lanes
2. Curb and gutter, part of which is used as part of the parking lane
3. Pervious area between curb and sidewalk
4. Sidewalk
5. Pervious area between sidewalk and property line.
6. One half of an alley in some neighborhoods
7. Part of driveway

How Imperviousness Varies for Different Types of Urban Developments
Neighborhoods are the heart of urban development and the objective is to develop sustainable neighborhoods. Commercial, industrial and public areas can be part of the neighborhood or separate entities. For the purposes of this discussion, three categories of 20th century neighborhoods are defined: pre-automobile, pre-expressway automobile, and post-expressway automobile. The general attributes of these categories are shown in Table 2-6.

Pre-automobile neighborhoods were laid out and developed prior to 1920 and did not include accommodation of the automobile as an important design factor. With automobile use becoming significant in urban areas during the period from the 1920's to 1950's, the federal government encouraged the development of suburban type subdivisions with driveways and garages. The massive federally supported urban expressway program began in the late 1950s and now affects virtually every major community in the United States. The availability of expressways and the provision of "free" parking at destination points greatly accelerated the trend towards individual automobile travel in cities and surrounding areas. The term "automobile" is used to cover all categories of personal motor vehicles.

Table 2-6. Attributes of 20th century neighborhoods in the U.S.

	Pre-Automobile	Pre-expressway	Post-expressway
Neighborhoods			
Population Density	High	Medium	Low
Street Connectivity	High	Medium	Low
Alleys	Typical	Rarer	Very rare
Driveways	Rare	Some	Typical
Parking	On-street	On and off street	Mainly off-street
Dwelling Unit Size	Smaller	Medium	Larger
Garages	No	One car	Two-three car
Cars/dwelling unit	0	1	1-4
People/dwelling unit	4-5	3-4	2-3
VMT/cap-year	Negligible	2,000-3,000	8,000-10,000
Sidewalks	Yes	Yes	Yes
Type of sewer system	Combined	Mixed	Separate
Pervious areas/dwelling unit	Low	Medium	High
Land uses	Mixed	Hybrid	Separated
Covered porches	Very popular	Less popular	Less popular
Patios	Rare	More popular	Very popular
Commercial	Neighborhood/ Strip	Strip development	Shopping Center
Industrial	Neighborhood/ Separate	Neighborhood/ Separate	Separate

Pre-Automobile Neighborhoods

The approach taken is to evaluate a variety of residential land use patterns at the block or subdivision level and to vary the housing density for these units in order to calculate how directly connected (DCIA) and other (OIA) imperviousness varies as land use changes. A standard gridiron block with data from Chicago, IL and Boulder, CO is used. Two standard Chicago blocks are shown in Figure 2-10 (APWA 1968). This five acre block contains 36 houses (popularly called bungalows in Chicago) within the five acre block or an overall average density of 7.2 dwelling units per gross acre. Because of the high density and soils with limited infiltration capacity, the downspouts from the rooftops are connected directly to the sewers. The total imperviousness is about 57%. The DCIA is about 40% with the houses contributing about one half of the DCIA.

Land use in an older neighborhood in Boulder, CO is shown in Figure 2-10. The block size is identical to the Chicago blocks, (i.e., five acres in area) with a length of 660 feet and a width of 330 feet. However, unlike the homogeneous lot and house sizes in Chicago, the Boulder lots and houses vary widely in size and shape. The alleys in Boulder are semi-improved.

A spreadsheet was set up to estimate the nature of the imperviousness for these traditional gridiron street patterns. Six different housing densities are placed on these five acre blocks ranging from a high of 14.2 to a low of 2.4 dwelling units per gross acre. All lot sizes are identical within a given category. The results are shown in Tables 2-7 for total imperviousness and 2-8 for directly connected imperviousness.

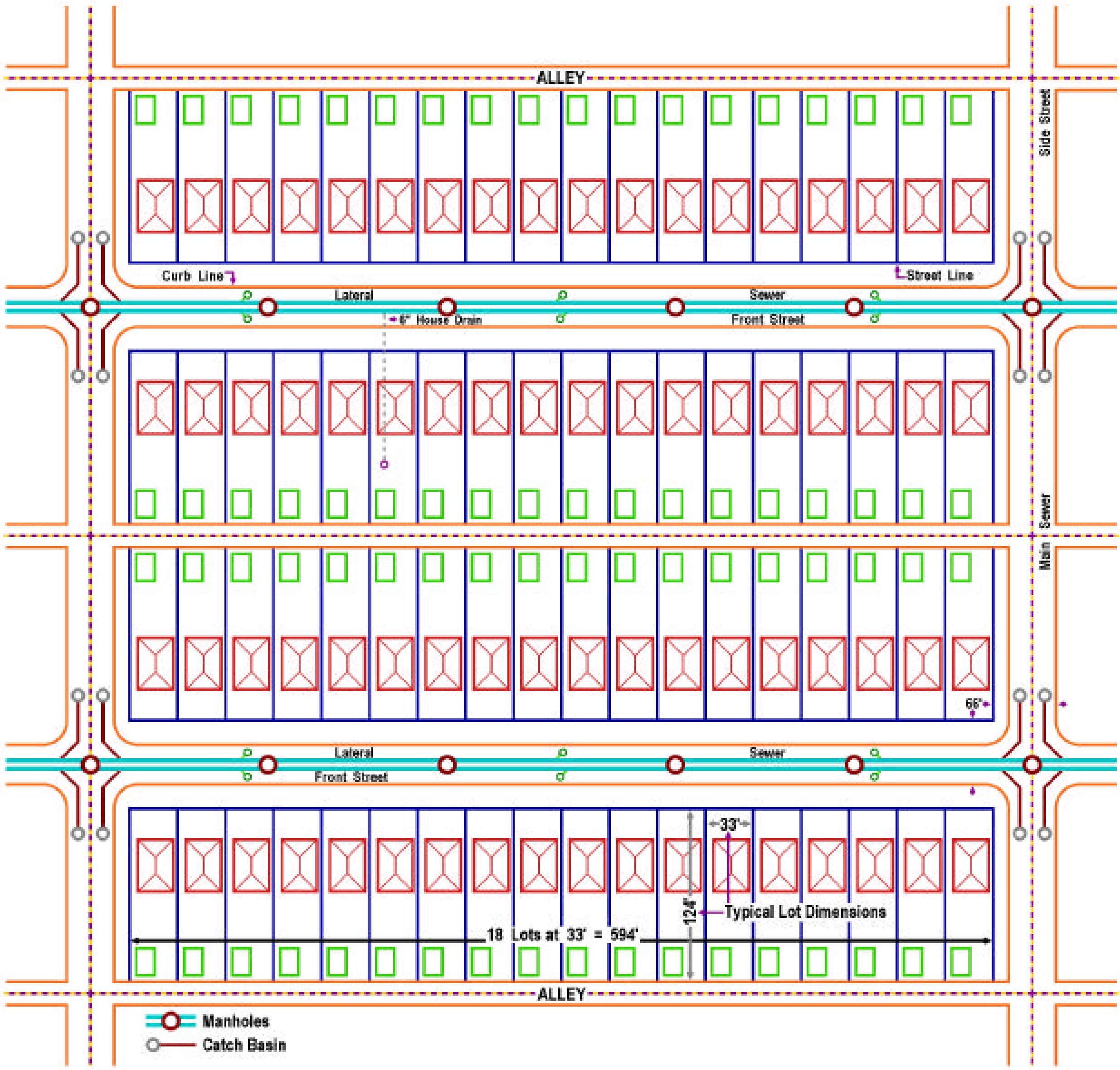


Figure 2-9. Typical unit residential area, Chicago, IL (APWA, 1968).

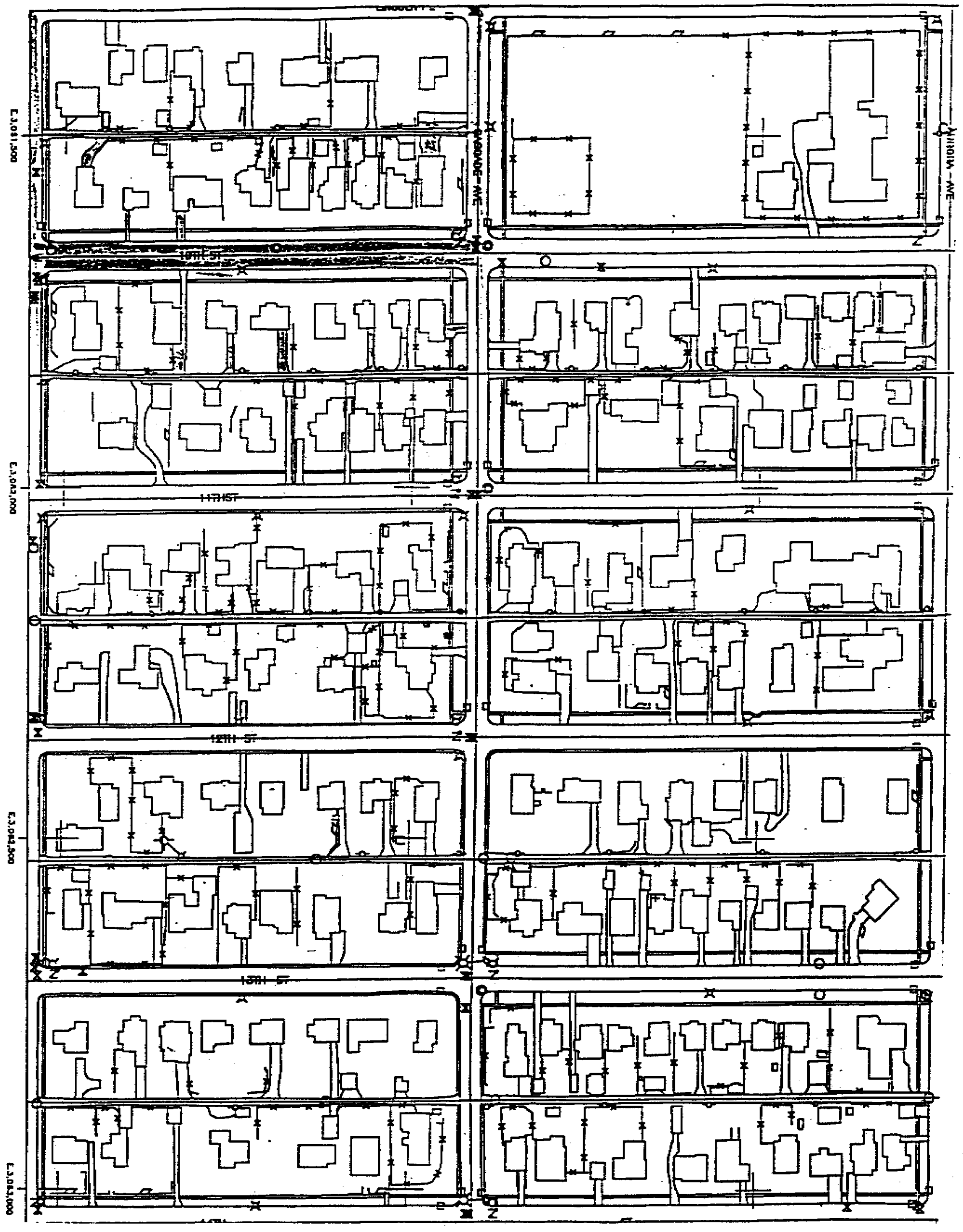


Figure 2-10. Aerial view of 10 blocks in an older neighborhood in Boulder, CO.

Table 2-7. Attributes of dwelling units located on traditional grid street network-total imperviousness**.**

Description	Dwelling Units/ Block	Living Area sq. ft./DU	Dwelling Units/Lot	Footprint Living Area sq. ft./DU	Garage Roof sq. ft./DU	Driveway sq. ft./DU	Street sq. ft./DU	Walkways sq. ft./DU	Alley sq. ft./DU	Transport sq. ft./DU	Total Impervious Area sq. ft./DU	Total Pervious Area sq. ft./DU	Total Area sq. ft./DU	% Total Impervious Area	Imp. Area Transport/ Living	Areas for Vehicles		
																Parking sq. ft./DU	Traffic sq. ft./DU	Total sq. ft./DU
1. Original inner city houses	48	800	1	800	160	50	701	716	209	1,836	2,636	1,902	4,538	58.1%	2.29	526	386	911
2. Larger bungalows	36	1,170	1	1,170	320	50	935	781	278	2,364	3,534	2,516	6,050	58.4%	2.02	791	514	1,306
3. Two flats-two story	72	1,170	2	585	160	25	468	391	139	1,182	1,767	1,258	3,025	58.4%	2.02	395	257	653
Mean	52	1,047	1.33	852	213	42	701	629	209	1,794	2,646	1,892	4,538	58.3%	2.11	571	386	956
4. Houses w. side driveway/rear garage	24	1,500	1	1,500	320	800	1,403	912	0	3,434	4,934	4,141	9,075	54.4%	2.29	1,751	771	2,523
5. Houses with driveway/garage in front.	20	2,100	1	2,100	320	600	1,683	990	0	3,593	5,693	5,197	10,890	52.3%	1.71	1,677	926	2,603
6. Low density house	12	2,400	1	2,400	320	600	2,806	1,303	0	5,028	7,428	10,722	18,150	40.9%	2.10	2,182	1,543	3,725
Mean	19	2,000	1	2,000	320	667	1,964	1,068	0	4,019	6,019	6,687	12,705	49.2%	2.03	1,870	1,080	2,950

Table 2-8. Attributes of dwelling units located on traditional grid street network-directly connected imperviousness**.**

Description	Dwelling Units/ Block	Living Area sq. ft./DU	Dwelling Units/Lot	Footprint Living Area sq. ft./DU	Garage Roof sq. ft./DU	Driveway sq. ft./DU	Street sq. ft./DU	Walkways sq. ft./DU	Alley sq. ft./DU	Transport sq. ft./DU	DC Impervious Area sq. ft./DU	Pervious Area sq. ft./DU	Total Area sq. ft./DU	% DC Impervious Area	DCIA Transport/ Living	Areas for Vehicles		
																Parking sq. ft./DU	Traffic sq. ft./DU	Total sq. ft./DU
1. Original inner city houses	48	800	1	800	40	25	701	179	209	945	1,745	2,792	4,538	38.5%	1.18	381	386	766
2. Larger bungalows	36	1,170	1	1,170	80	25	935	195	278	1,235	2,405	3,645	6,050	39.8%	1.06	526	514	1,040
3. Two flats-two story	72	1,170	2	585	80	13	468	98	139	658	1,243	1,782	3,025	41.1%	1.12	303	257	560
Mean	52	1,047	1.33	852	67	21	701	157	209	946	1,798	2,740	4,538	39.8%	1.12	403	386	789
4. Houses w. side driveway/rear garage	24	750	1	750	80	200	1,403	228	0	1,910	2,660	6,415	9,075	29.3%	2.55	911	771	1,683
5. Houses with driveway/garage in front.	20	693	1	693	80	150	1,683	248	0	2,161	2,854	8,037	10,890	26.2%	3.12	987	926	1,913
6. Low density house	12	600	1	600	80	150	2,806	326	0	3,361	3,961	14,189	18,150	21.8%	5.60	1,492	1,543	3,035
Mean	38	890	1.19	779	72	83	1,242	204	119	1,602	2,381	5,657	8,038	33.8%	2.25	715	683	1,398

Notes;

DU-dwelling unit

Street area includes side streets at the end of the block.

Average Total Imperviousness/DU

Item	%	Sq. ft.
Living area	33.2%	2,000
Garage	5.3%	320
Driveway	11.1%	667
Alley	0.0%	0
Street	32.6%	1,964
Walkways	17.8%	1,068
Total	100.0%	6,019

Average Directly Connected Imperviousness/DU

Item	%	Sq. ft.
Living area	34.1%	890
Garage	2.8%	72
Driveway	3.2%	83
Alley	4.6%	119
Street	47.6%	1,242
Walkways	7.8%	204
Total	100.0%	2,611

Imperviousness in Pre-Automobile Era

Categories 1 to 3 in Tables 2-7 and 2-8 represent the pre-automobile era and are all served by alleys. Densities range from 5.2 to 14.4 dwelling units per acre. Garages are assumed to exist although they probably were used for other purposes and were called sheds. The total imperviousness for these three land uses is about 58% and the DCIA is about 40%. The rooftops are directly connected to the sewer because of the higher densities and lack of sufficient pervious areas to receive the roof runoff. The transition point at which roof runoff can be discharged onto pervious areas needs to be determined based on local conditions. Even for this pre-automobile condition, transportation related imperviousness is over twice the imperviousness caused by the living area. However, walkways (front, rear, and side) are a significant part of the transportation component.

Pre-Expressway Neighborhoods

Large-scale development began after World War II with communities such as Levittown, NY (Southworth and Ben-Joseph 1997). This residential street is typical of the design standards for suburban developments, (i.e., wide streets with curb and gutter, sidewalks on both sides of the street, paved driveways, and garages or carports). Most newer suburban communities followed federal street standards promulgated by FHA during the 1930's.

Results for Pre-Expressway Era

Cases 4 to 6 in Tables 2-8 and 2-9 represent developments that accommodate the automobile. The first phase of this transition was to eliminate alleys and construct side drives to garages in the rear of the house. Then, garages were attached directly to the house, and lastly the houses grew in size. The number of dwelling units per gross acre ranges from 2.4 to 4.8. The declining dwelling unit densities reduced total imperviousness to 41 to 54%, less than traditional developments, but not proportionately less. The DCIA ranges from 22 to 29%, a significant decrease from 38 to 41% associated with earlier developments. The major reduction in DCIA is due to disconnecting roof downspouts and eliminating alleys. However, the DCIA area per dwelling unit increases substantially from an average of about 1,800 to 3,200 square feet due to the larger garages, driveways, and lot sizes.

Post-Expressway Neighborhoods

The availability of expressways allowed people to move even farther from the core urban areas. The major impact of the expressways is the need for more vehicles per family and with cheaper land and increased economic prosperity associated with a healthy economy and the trend towards two working parents, house and lot sizes continued to grow. Thus, contemporary houses have larger garages and driveways, and more street frontage per house. A sample of 24 contemporary homes taken from Sunset (1992) was used to evaluate the expected nature of imperviousness in contemporary housing. The sample consisted of 13 single story houses and 11 two story houses.

One Story Houses: The results for the single story houses are shown in Tables 2-9 and 2-10 for total imperviousness and DCIA, respectively. No explicit street pattern is assumed for this development. Thus, the street and sidewalk areas are underestimated, probably by 10-15%. Development densities range from 2.0 to 5.4 houses per acre. The results indicate that total imperviousness is relatively insensitive to housing density and ranges from 36 to 48%. Total imperviousness actually increases as dwelling unit density decreases due to larger garages, longer driveways and more street length per house. On the average, the living area constitutes 41% of the total imperviousness, but only 22% of the DCIA. Thus, the transportation component dominates as the primary source of total, and more importantly, directly connected imperious area.

Measured in absolute terms in terms of total impervious area per house, the results indicate that total impervious area per house increases from about 4,000 square feet to almost 8,700 square feet as the living area goes from 1,272 square feet to 4,284 square feet. Parking is responsible for most of the total impervious area for vehicles, an average of 2,041 square feet of parking compared to an average of 811 square feet for traffic movement. Only about half of the impervious area for parking is directly connected. Thus, its impact is lessened. Overall, streets constitute over 61% of the DCIA. The street is used both for parking and traffic flow.

Two-story Houses: The results for the two story houses are shown in Tables 2-11 and 2-12 for total imperviousness and DCIA, respectively. No explicit street pattern is assumed for this development. Thus, the street and sidewalk areas are underestimated, probably by 10-15%. Development densities range from about 2.9 to 6.9 houses per acre. The results indicate that total imperviousness is relatively insensitive to housing density and ranges from 31 to 80%. Total imperviousness actually increases as dwelling unit density decreases due to larger garages, longer driveways and more street length per house. On the average, the living area constitutes 37% of the total imperviousness, but only 20% of the DCIA. As before, the transportation component dominates as the primary source of total and more importantly, directly connected imperious area.

Measured in absolute terms in terms of total impervious area per house, the results indicate that total impervious area per house increases from about 2,800 square feet to almost 6,376 square feet as the living area goes from 1,193 square feet to 3,728 square

Table 2-9. Attributes of dwelling units located on traditional grid street network-total imperviousness.

Description	Dwelling Units/ Block	Living Area sq. ft./DU	Dwelling Units/Lot	Footprint Living Area sq. ft./DU	Garage Roof sq. ft./DU	Driveway sq. ft./DU	Street sq. ft./DU	Walkways sq. ft./DU	Alley sq. ft./DU	Transport sq. ft./DU	Total Impervious Area sq. ft./DU	Total Pervious Area sq. ft./DU	Total Area sq. ft./DU	% Total Impervious Area	Imp. Area Transport/ Living	Areas for Vehicles Parking sq. ft./DU	Areas for Vehicles Traffic sq. ft./DU	Areas for Vehicles Total sq. ft./DU
1. Original inner city houses	48	800	1	800	160	50	701	716	209	1,836	2,636	1,902	4,538	58.1%	2.29	526	386	911
2. Larger bungalows	36	1,170	1	1,170	320	50	935	781	278	2,364	3,534	2,516	6,050	58.4%	2.02	791	514	1,305
3. Two flats-two story	72	1,170	2	585	160	25	468	391	139	1,182	1,767	1,258	3,025	58.4%	2.02	395	257	653
Mean	52	1,047	1.33	852	213	42	701	629	209	1,794	2,646	1,892	4,538	58.3%	2.11	571	386	956
4. Houses w. side driveway/rear garage	24	1,500	1	1,500	320	800	1,403	912	0	3,434	4,934	4,141	9,075	54.4%	2.29	1,751	771	2,523
5. Houses with driveway/garage in front.	20	2,100	1	2,100	320	600	1,683	990	0	3,593	5,693	5,197	10,890	52.3%	1.71	1,677	926	2,603
6. Low density house	12	2,400	1	2,400	320	600	2,805	1,303	0	5,028	7,428	10,722	18,150	40.9%	2.10	2,182	1,543	3,725
Mean	19	2,000	1	2,000	320	667	1,964	1,068	0	4,019	6,019	6,687	12,705	49.2%	2.03	1,870	1,080	2,950

Table 2-10. Attributes of dwelling units located on traditional grid street network-directly connected imperviousness.

Description	Dwelling Units/ Block	Living Area sq. ft./DU	Dwelling Units/Lot	Footprint Living Area sq. ft./DU	Garage Roof sq. ft./DU	Driveway sq. ft./DU	Street sq. ft./DU	Walkways sq. ft./DU	Alley sq. ft./DU	Transport sq. ft./DU	DC Impervious Area sq. ft./DU	Pervious Area sq. ft./DU	Total Area sq. ft./DU	% DC Impervious Area	DCIA Transport/ Living	Areas for Vehicles Parking sq. ft./DU	Areas for Vehicles Traffic sq. ft./DU	Areas for Vehicles Total sq. ft./DU
1. Original inner city houses	48	800	1	800	40	25	701	179	209	945	1,745	2,792	4,538	38.5%	1.18	381	386	766
2. Larger bungalows	36	1,170	1	1,170	80	25	935	195	278	1,235	2,405	3,645	6,050	39.8%	1.06	526	514	1,040
3. Two flats-two story	72	1,170	2	585	80	13	468	98	139	658	1,243	1,782	3,025	41.1%	1.12	303	257	560
Mean	52	1,047	1.33	852	67	21	701	157	209	946	1,798	2,740	4,538	39.8%	1.12	403	386	789
4. Houses w. side driveway/rear garage	24	750	1	750	80	200	1,403	228	0	1,910	2,660	6,415	9,075	29.3%	2.55	911	771	1,683
5. Houses with driveway/garage in front.	20	693	1	693	80	150	1,683	248	0	2,161	2,854	8,037	10,890	26.2%	3.12	987	926	1,913
6. Low density house	12	600	1	600	80	150	2,805	326	0	3,361	3,961	14,189	18,150	21.8%	5.60	1,492	1,543	3,035
Mean	38	690	1.19	779	72	83	1,242	204	119	1,602	2,381	5,657	8,038	33.8%	2.25	715	683	1,398

Notes;
DU-dwelling unit
Street area includes side streets at the end of the block.

Average Total Imperviousness/DU

Item	%	Sq. ft.
Living area	33.2%	2,000
Garage	5.3%	320
Driveway	11.1%	667
Alley	0.0%	0
Street	32.6%	1,964
Walkways	17.8%	1,068
Total	100.0%	6,019

Average Directly Connected Imperviousness/DU

Item	%	Sq. ft.
Living area	34.1%	890
Garage	2.8%	72
Driveway	3.2%	83
Alley	4.6%	119
Street	47.6%	1,242
Walkways	7.8%	204
Total	100.0%	2,611

Table 2-11. Attributes of thirteen contemporary one story houses-total imperviousness.

Number	% DCIA Living Area sq. ft.	25.00% Living+ porch/storage sq. ft.	25.00% Garage Roof sq. ft.	25.00% Driveway sq. ft.	100.00% Street sq. ft.	25.00% Walkways sq. ft.	Transport sq. ft.	Impervious Total sq. ft.	Pervious Total sq. ft.	Grand Total sq. ft.	% Total Impervious Area	Imp. Area Transport/ Living	Area for vehicles Parking sq. ft.	 Traffic sq. ft.	 Total sq. ft.
1	1,272	1,272	462	640	1,309	285	2,696	3,968	5,888	9,856	40.3%	2.12	1,718	693	2,411
2	1,283	1,300	400	600	1,224	260	2,484	3,784	4,784	8,568	44.2%	1.91	1,576	648	2,224
3	1,300	1,560	441	880	1,173	245	2,739	4,299	4,671	8,970	47.9%	1.76	1,873	621	2,494
4	1,418	1,668	484	800	1,411	315	3,010	4,678	5,033	9,711	48.2%	1.80	1,948	747	2,695
5	1,428	1,478	400	400	1,139	235	2,174	3,652	4,388	8,040	45.4%	1.47	1,336	603	1,939
6	1,458	1,458	400	880	1,309	285	2,874	4,332	5,601	9,933	43.6%	1.97	1,896	693	2,589
7	1,689	1,689	576	500	1,190	250	2,516	4,205	4,685	8,890	47.3%	1.49	1,636	630	2,266
8	2,000	2,000	672	900	1,428	320	3,320	5,320	8,036	13,356	39.8%	1.66	2,244	756	3,000
9	2,180	2,280	529	600	1,496	340	2,965	5,245	8,659	13,904	37.7%	1.30	1,833	792	2,625
10	2,400	2,660	550	1,000	2,006	490	4,046	6,706	11,938	18,644	36.0%	1.52	2,494	1,062	3,556
11	2,968	3,208	450	1,200	2,040	500	4,190	7,398	13,002	20,400	36.3%	1.31	2,610	1,080	3,690
12	3,735	3,935	768	800	2,210	550	4,328	8,263	12,537	20,800	39.7%	1.10	2,608	1,170	3,778
13	4,284	4,384	630	1,200	1,989	485	4,304	8,688	13,542	22,230	39.1%	0.98	2,766	1,053	3,819
Mean	2,109	2,222	520	800	1,533	351	3,204	5,426	7,905	13,331	42.0%	2	2,041	811	2,853

Table 2-12. Attributes of thirteen contemporary one story houses-directly connected imperviousness.

Number	Living Area sq. ft.	Living+ porch/storage sq. ft.	Garage Roof sq. ft.	Driveway sq. ft.	Street sq. ft.	Walkway sq. ft.	Transport sq. ft.	DC Imperv. Total sq. ft.	Perv. Total sq. ft.	Grand Total sq. ft.	% DCIA	Imp. Area Transport/ Living	Area for vehicles Parking sq. ft.	 Traffic sq. ft.	 Total sq. ft.
1	1,272	318	116	160	1,309	71	1,656	2,928	6,928	9,856	29.7%	5.2	892	693	1,585
2	1,283	325	100	150	1,224	65	1,539	2,822	5,746	8,568	32.9%	4.7	826	648	1,474
3	1,300	390	110	220	1,173	61	1,565	2,865	6,106	8,970	31.9%	4.0	882	621	1,503
4	1,418	417	121	200	1,411	79	1,811	3,229	6,482	9,711	33.2%	4.3	985	747	1,732
5	1,428	370	100	100	1,139	59	1,398	2,826	5,214	8,040	35.1%	3.8	736	603	1,339
6	1,458	365	100	220	1,309	71	1,700	3,158	6,775	9,933	31.8%	4.7	936	693	1,629
7	1,689	422	144	125	1,190	63	1,522	3,211	5,680	8,890	36.1%	3.6	829	630	1,459
8	2,000	500	168	225	1,428	80	1,901	3,901	9,455	13,356	29.2%	3.8	1,065	756	1,821
9	2,180	570	132	150	1,496	85	1,863	4,043	9,861	13,904	29.1%	3.3	986	792	1,778
10	2,400	665	138	250	2,006	123	2,516	4,916	13,728	18,644	26.4%	3.8	1,332	1,062	2,394
11	2,968	802	113	300	2,040	125	2,578	5,546	14,855	20,400	27.2%	3.2	1,373	1,080	2,453
12	3,735	984	192	200	2,210	138	2,740	6,475	14,326	20,800	31.1%	2.8	1,432	1,170	2,602
13	4,284	1,096	158	300	1,989	121	2,568	6,852	15,378	22,230	30.8%	2.3	1,394	1,053	2,447
Mean	2109	556	130	200	1,533	88	1,950	4,059	9,272	13,331	31.1%	3.8	1,051	811	1,863

Total Imperviousness		
Living area	40.96%	2222
Garage	9.59%	520
Driveway	14.74%	800
Street	28.25%	1533
Sidewalk	6.46%	351
Total	0.00%	5426

DC Imperviousness		
Living area	22.17%	556
Garage	5.19%	130
Driveway	7.98%	200
Street	61.16%	1533
Sidewalk	3.50%	88
Total	100.00%	2506

feet. Most of the total impervious area for vehicles is for parking, an average of 1,725 square feet of parking compared to an average of 662 square feet for traffic movement. Only about half of the impervious area for parking is directly connected. Thus, its impact is lessened. Overall, streets constitute over 63% of the DCIA. The street is used both for parking and traffic flow.

General Conclusions Regarding the Effect of Changing Land Use

Three 20th century land use patterns: pre-automobile, pre-expressway, and post-expressway, were evaluated. The major trend over the century has been towards decreased development densities. Densities greater than about eight dwelling units per acre are difficult to achieve with automobiles since insufficient parking by contemporary standards is available. Therefore, the earlier impact of the automobile was to retrofit existing neighborhoods and foster growth in nearby suburbs that could accommodate automobiles as a major user of land. The development of expressways allowed people to move even farther out of the core urban areas. This movement resulted in even more dependence on automobiles and led to even lower development densities. Thus, the overall results of the above analysis can be captured by showing the effect of density on infrastructure utilization. The results are summarized below.

Higher densities significantly reduce the lengths of streets, water mains, sanitary and storm sewers needed per dwelling unit as shown in Table 2-13 and Figure 2-11 for the five acre block studied as part of traditional developments. The general equation for feet of street per dwelling unit for this five acre case is:

$$L = \frac{198}{DUD} \qquad \text{Equation 2-1}$$

where *L* = feet of street per dwelling unit, and
DUD = dwelling units per gross acre.

The length shown in Equation 2-1 consists of one half of the street frontage per dwelling unit plus a prorated share of the side street length. Urban sprawl is considered to be lot densities of three per acre or less. As indicated by Figure 2-11 , the street length per dwelling unit increases rapidly at lower densities reaching 100 feet per dwelling unit at two units per acre, four times the length at eight units per acre. This length per dwelling unit is a critical parameter because the street, water main, sanitary sewer, and storm sewer lengths all increase in the same proportion.

The service area per household increases according to the same type of relationship as for infrastructure length, that is:

$$A = \frac{43560}{DUD} \qquad \text{Equation 2-2}$$

where *A* = square feet of area per dwelling unit, and
DUD = dwelling units per gross acre.

Table 2-13: Relationship between street length and dwelling unit density for a five acre rectangular block of dimensions 660 feet by 330 feet.

DUD Dwelling Unit Density (dwelling units/acre)	Street Length Per Dwelling Unit (feet)
2	99.0
3	66.0
4	49.5
5	39.6
6	33.0
7	28.3
8	24.8
9	22.0
10	19.8
11	18.0
12	16.5
13	15.2
14	14.1
15	13.2

Figure 2-11. Relationship between street length and dwelling unit density for a five acre rectangular block of dimensions 660 feet by 330 feet.

The results are shown in Table 2-14 and Figure 2-12. Lot area per dwelling unit is also a critical parameter in determining infrastructure costs. Larger lots generate an increased demand for lawn watering, the largest source of variability in urban water supply.

Another significance of lot area is that storm sewer peak design flows for small catchments are typically calculated using the Rational formula,

$$Q = CiA \qquad \text{Equation 2-3}$$

where Q = peak discharge rate,
C = runoff coefficient that depends on the land use,
i = rainfall intensity, and
A = drainage area.

Q increases linearly with drainage area in Equation 2-3. The only offsetting factor is if the runoff coefficient decreases as A increases. The runoff coefficient is often assumed to equal the imperviousness as shown in Figure 2-13. Using a database of DUD as a function of total and DCIA developed as part of this study, a relationship between imperviousness and DUD was derived. The results, shown in Figure 2-14, indicate that total imperviousness decreases from about 60% at a DUD of 10 to about 40% at a DUD of two. The net effect, shown in Table 2-15 is more than a three-fold increase in *CA* and, therefore, peak discharge rate, as densities decrease from 10 to two DU/gross acre.

Table 2-14. Effect of dwelling unit density on CA in the Rational formula

DUD Dwelling Unit Density (dwelling units/acre)	**A Lot Area Per Dwelling (sq. ft.)**	**I Imperviousness (%)**	**CA In (sq. ft.)**
2	21,780	40	8,712
10	4,356	60	2,616

The preceding results imply that serving contemporary lower density residential developments is significantly more expensive per dwelling unit than it is for higher density developments. Is this cost reflected in the charges for services rendered? If the new users paid system development charges (SDC) that covered the cost of the local improvements, then a significant part of this added cost is equitably assigned. Most of the charges for water supply are assessed based on water use. Per capita indoor water use is fairly constant. However, outdoor water use depends on the demand for irrigation water which ranges from insignificant in the northeastern U.S. to dominant in the arid southwestern U.S. If irrigation is not a significant water use and SDC's were not assessed, then the lower density developments are being subsidized since they require more piping per unit of water delivered. If irrigation is significant, then the equity of the charges depends on the charge for outdoor water use. Wastewater charges are either fixed per household or assessed based on indoor water use. This charging procedure is unfair to people living in higher density areas since they use less piping per family. Stormwater charges are a fixed amount per month, or are based on impervious area. Only in the latter case are charges assessed in proportion to the contribution to the problem.

Table 2-15: Relationship between dwelling unit density and area per lot.

DUD (Dwelling Unit Density) (dwelling units/acre)	Lot Area (sq. ft.)
2	21,780
3	14,520
4	10,890
5	8,712
6	7,260
7	6,223
8	5,445
9	4,840
10	4,356
11	3,960
12	3,630
13	3,351
14	3,111
15	2,904

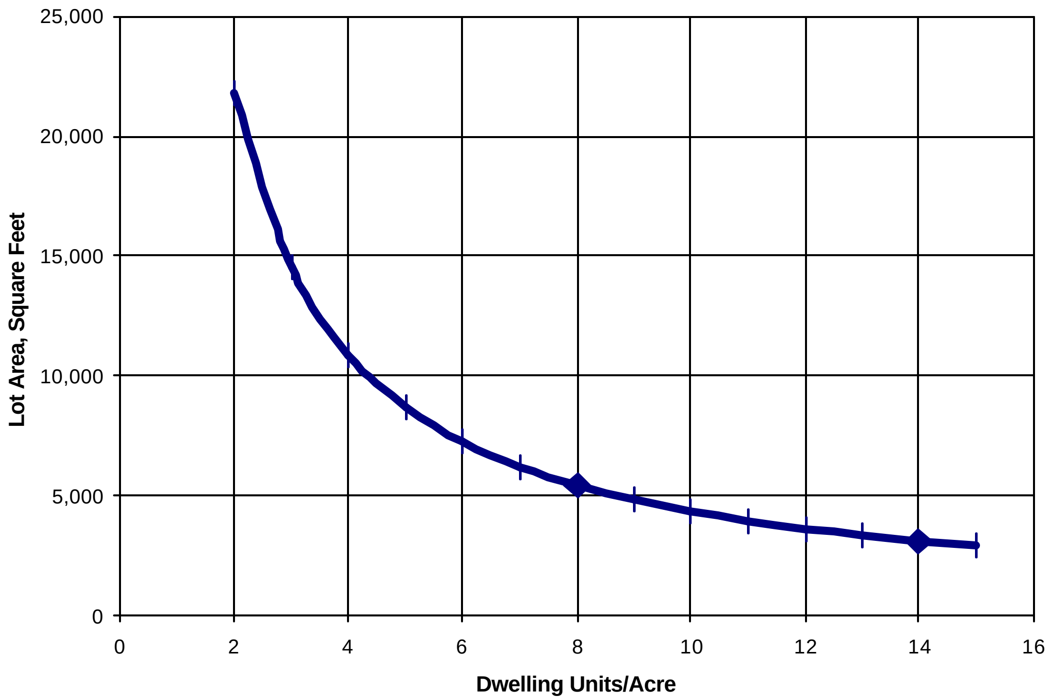


Figure 2-12. Relationship between dwelling unit density and area per lot.

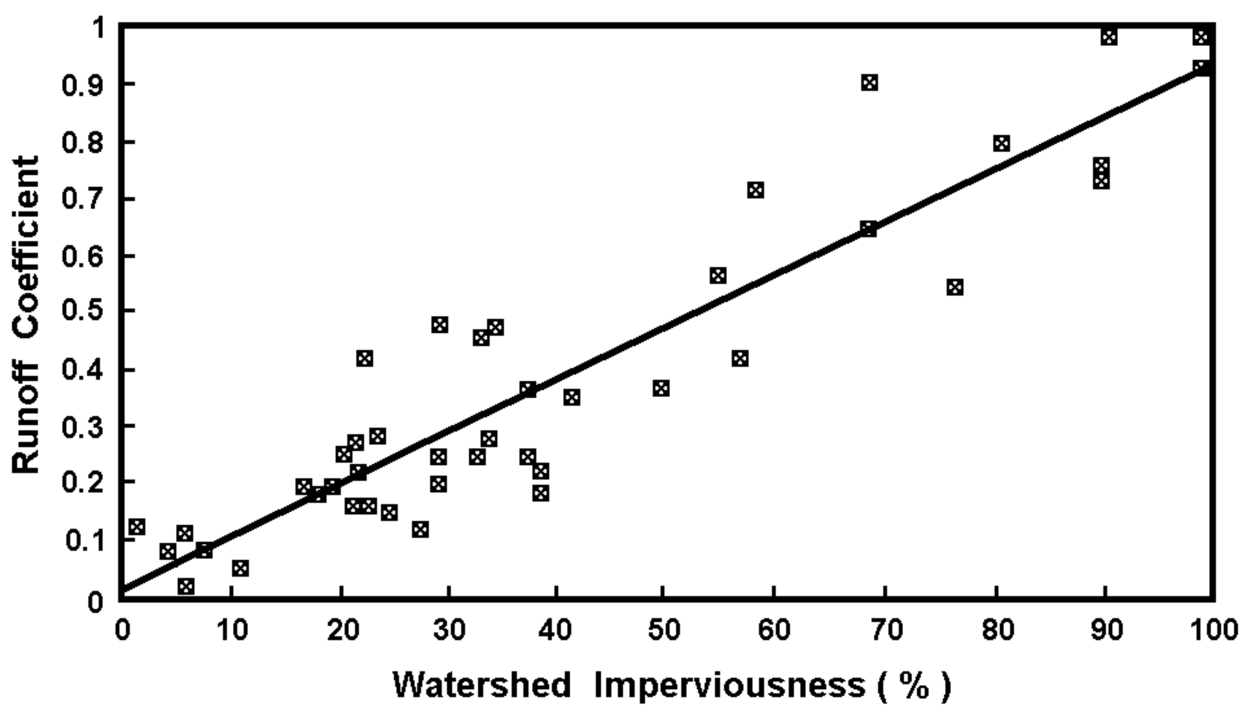


Figure 2-13. Watershed imperviousness and the storm runoff coefficient (WEF/ASCE 1998).

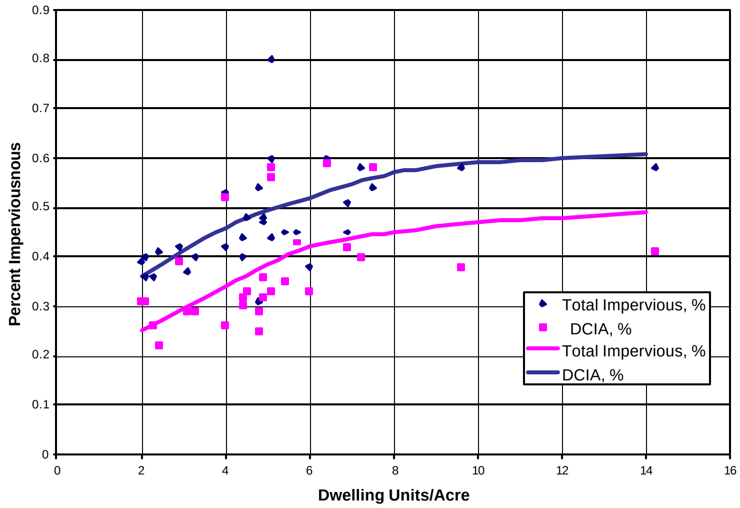


Figure 2-14. Effect of dwelling unit density on imperviousness.

In summary, overall dwelling unit density is a good measure of the impact of residential development on infrastructure. Densities above about eight dwelling units per acre are difficult to achieve in areas that are dependent on the automobile for transportation since there is insufficient space to accommodate the automobile with existing land use zoning requirements.

The quantity of stormwater runoff per person has grown dramatically during the past century. The following factors are the major causes of this growth:

1. The introduction of automobiles into cities: Automobiles are very inefficient people movers in cities with regard to the space and generation of pollutants. A vehicle weighing 2,500 to 4,000 pounds is used to carry a 150 pound person around the city. This vehicle is only used about 1-5% of the time. When not in use, it must be parked. Each off-street parking space uses 300-400 square feet of impervious area. In residential areas, transportation related imperviousness accounts for over 65% of total imperviousness and nearly 80% of the DCIA. Within residential neighborhoods alone, about 1.25 to 2.0 square feet of impervious area is generated for transportation for every square foot of living area. Similar ratios exist for commercial areas.

2. The trend towards larger houses: House sizes have grown significantly in the past 40 years from about 1,000 square feet to over 2,000 square feet as families move to outlying areas.

3. The trend towards larger lots: Lot sizes have also grown significantly as families provide recreation and open space on each lot as opposed to using common

areas. Lot sizes have also had to grow to accommodate larger garages and driveways.

4. The trend towards smaller families: Smaller family sizes and larger houses cause the need for support infrastructure per capita to increase accordingly.

5. The green trend of providing more open space as part of the development: This open space further reduces densities and increases sprawl. Properly designed, some or all of this open space could provide essential water infrastructure functions such as stormwater retention.

Given that demands for stormwater management have increased dramatically due to the pervasive influence of the automobile, the trend towards lower density sprawl development, and the desire for open space, can any of these patterns be changed? The individual sources of imperviousness and their nature are discussed in the following sections.

Components of Urban Land Use and Stormwater Problems

The components of urban land use are examined in this section. For each component, the relative importance as a source of stormwater quantity and quality problems is discussed. The controllability of stormwater from each component is then analyzed.

Streets and Highways

Urban street patterns have changed during the 20th century, with the automobile having a major influence on street design at all levels. Southworth and Ben-Joseph (1995) summarize this evolution. They trace the major change in philosophy for street design to the 1930s when the federal government became involved in developing guidelines for subdivisions as part of its program to insure home mortgages. The traditional pattern is the gridiron with typical block dimensions of 1/8 by 1/16 of a mile as was shown earlier. The most radical departure from this pattern was the Radburn development in New Jersey that used narrower streets in the neighborhood.

In 1936, the Federal Housing Administration (FHA) rejected the grid pattern for residential neighborhoods, and has continued this policy of preferring other street layouts (Southworth and Ben-Joseph 1995). Their primary reasons for rejecting the gridiron pattern are:

1. It requires more paved area than necessary because all residential streets are built to the same specifications.

2. It requires more expensive type of pavement since the traffic is dispersed throughout the neighborhood and thus the streets must be designed to a higher standard.

3. This heavier traffic demand creates a hazard.

4. The gridiron layout is monotonous and uninteresting.

The FHA recommended a hierarchical street pattern. For residential streets, they recommended curvilinear alignments, cul-de-sacs, and courts. Desirable design criteria promulgated by the FHA included (Southworth and Ben-Joseph 1995):

1. Layout should discourage through traffic.

2. Minimum width of a residential street should be 50 feet with 24 feet of pavement, eight foot planting/utility strips and four foot walks.

3. Cul-de-sacs are the most attractive street layout for family dwellings.

4. Minimum setbacks for streets should be 15 feet.

5. Front yard should avoid excessive planting, for a more pleasing and unified effect along the street.

These early FHA guidelines had a tremendous influence on residential development in the United States because of their financial leverage over developers and home buyers. The Institute of Transportation Engineers (ITE) has also had a major influence on residential street design. Their perspective is heavily influenced by traffic flow and parking considerations. They recommend (Southworth and Ben-Joseph 1995):

1. Right of way minimum of 60 feet.

2. Pavement width of 32-34 feet.

3. Cul-de-sacs should have a maximum length of 1,000 feet with a 50-foot radius at the end.

4. Parking lanes should be 8 feet in width.

The influence of these street design standards on drainage and stormwater quality does not seem to have been a significant factor in the decision making process.

The American Association of State Highway and Transportation Officials (AASHTO) has been responsible for developing the design standards for highways and streets. The primary reference is A Policy on Geometric Design of Highways and Streets (AASHTO 1984).

According to Khisty (1990), 10-13 foot lane widths predominate in the United States with 12 feet being the most common. The use of 11 foot lane widths is acceptable in urban areas due to higher right-of-way costs. Ten-foot lane widths are only acceptable on low speed urban streets.

Ewing (1996) divides residential streets into the categories of arterial, collector/sub-collector, and access. Four types of residential streets (i.e., non-arterials) exist. They are:

1. Collector
2. Sub-collector
3. Access-looped
4. Access-dead end

Southworth and Ben-Joseph (1997) provide a history of urban streets, a critique on current practices, and project the expected nature of streets in urban areas. They estimate that, worldwide, more than one third of all developed urban land is devoted to roads, parking lots, and other automobile infrastructure. In the urban U.S., about one half of the land is used for this purpose. In automobile oriented cities like Los Angeles, the percentage increases to two thirds (Hanson 1992, Renner 1988). These estimates are compatible with the results presented in the previous section.

Traditional gridiron street patterns were rejected as bad practice beginning in the 1930's based on recommendations from the federal government. They are enjoying a comeback as part of the interest in the new urbanism. Chellman (1997) provides a current summary of the pros and cons of traditional streets for neighborhoods. Features of traditional streets include a high degree of connectivity that maximizes mobility for non-motorists.

Transportation engineers tend to design streets to maximize convenience for the automobile subject to safety constraints. Recently, designers have attempted to recast the purpose of streets as multi-purpose components of the community with much more of a pedestrian orientation. Shared streets provide a multi-purpose use of residential streets. These streets have gained favor internationally but have not yet gained widespread acceptance in the U.S. Key impediments in the U.S. include dependency on automobiles, and concerns of liability if existing street standards are changed. Portland, OR is one of the few cities in the U.S. that is rethinking its approach to residential streets with its skinny streets program (Southworth and Ben-Joseph 1997). They have reduced street widths to 20-26 feet and have installed many traffic calming devices.

Streets have the potential to play a major role in stormwater management. Walesh (1989, Chapter 5) presents an analysis of the ability of a typical urban street, with curb and gutter, to temporarily convey or store stormwater runoff from major runoff events. Skokie, IL implemented an innovative approach to its streets by using them to intentionally convey and store stormwater in a controlled fashion so that combined sewers do not surcharge and back up into basements (Walesh and Carr, 1998).

Stormwater control is achieved in this cost-effective system using on-street berms coupled with catch basin flow regulators and, where needed, subsurface tanks.

Street Classification and Utilization

The Federal Highway Administration (FHWA) tabulates a variety of street related statistics that can be obtained on the internet at *http:/www.bts.gov/cgi-bin/stat/final_out.pl.* Results for urban areas in the United States are shown in Table 2-16. The major traffic carrying components of the highway system constitute only about 9% of the road mileage in urban areas. Local streets that carry little traffic constitute the bulk of the mileage, nearly 70%. Parking is allowed on the lesser used streets; thus, most of the parking is associated with local and collector streets. While the interstates, freeways, other expressways, and principal arterial streets constitute only 9.1% of the miles, they carry 58% of the traffic. At the other extreme, local streets, constituting 69.5% of the street length, carry only 13.8% of the traffic. Thus, in terms of managing imperviousness, the lesser used local streets are the prime candidates for evaluating whether they could be reduced in size.

The results of Table 2-16 also suggest that the primary sources of traffic related stormwater pollution are the intensively used street systems. This may suggest a control strategy of providing more treatment for these intensively used streets. This much smaller impervious area may be more amenable to control than trying to deal with the entire impervious area of the city.

Table 2-16: Street mileage in the U.S.

Urban	Miles of road	% of urban
Interstate	13,307	1.6%
Other freeways/expressways	9,022	1.1%
Other principal arterial	53,044	6.4%
Minor arterial	89,013	10.8%
Collector	87,918	10.6%
Local	574,119	69.5%
Total Urban	826,423	100.0%

Recommendations for Residential Streets

Southworth and Ben-Joseph (1997) recommend the following principles for future residential streets:

1. Support varied uses of residential streets including children’s play and adult recreation.

2. Design and manage street space for the comfort and safety of residents.
3. Provide a well-connected, interesting pedestrian network.
4. Provide convenient access for people who live on the street, but discourage through traffic; allow traffic movement, but do not facilitate it.
5. Differentiate streets by function.
6. Relate street design to the natural and historical setting.
7. Conserve land by minimizing the amount of land devoted to streets.

Contemporary texts on highway engineering do not deal with urban runoff problems. Khisty (1990) cautions of the need to evaluate air pollution and noise impacts as part of highway design. He doesn't mention highway runoff as a problem. Wright and Paquette (1996) describe conventional highway drainage design but do not discuss stormwater quality problems or the detrimental off-site impacts from highway runoff. The FHWA has sponsored several studies to address the issue of stormwater problems associated with highways. Young et al. (1996) present a detailed overview of highway runoff quality problems. For a more current view from FHWA on whether they consider highway runoff to be a serious problem, see http://www.tfhrc.gov/hnr20/runoff/runoff.html.

Streets and Stormwater Runoff

Whether residential streets are laid out in a grid-iron, curvilinear, or cul-de-sac format does not appear to have a major impact on the quantity of stormwater runoff per capita. The curvilinear and cul-de-sac layouts tend to have a larger impact per capita because of lower development densities. Schueler (1995) summarizes current national design standards for residential streets as shown in Table 2-17. Parking requires about eight feet of space and traffic lanes require about 10-12 feet per lane. Thus, streets with two way traffic and parking on both sides of the street would be 36 to 40 feet wide, if multi-purpose use is not incorporated in the design.

Average daily traffic (ADT) in vehicles per day is the common indicator of the utilization of streets for traffic. Schueler (1995) summarizes the expected traffic flow for various ADTs assuming 10 trips per dwelling unit per day and that the number of trips in the peak hour is 10% of the daily trips. The results are presented in Table 2-18 (Schueler 1995). As Schueler points out, for ADTs of 25 or less, it is reasonable to share parking and traffic lanes. Unfortunately, many cities have adopted regulations that require wide residential streets even in areas with little or no traffic.

Parking

The Institute of Transportation Engineers (ITE) recommends (Southworth and Ben Joseph, 1995) that on-street parking lanes should be eight feet in width and that driveway widths should be a minimum of 10 feet for one car, with a 20 foot-wide curb cut (five-foot flare on each end). According to Shoup (1995), off-street parking space per vehicle ranges from 300 to 350 square feet per space. This square footage includes the space itself, the access aisles, and the entry, exit area.

Table 2-17. Condensed summary of national design standards for residential streets (Schueler 1995).

Design Criteria	AASHTO	ITS	HEADWATER STREETS
Residential Street Categories	1	3, depending on use density	4, depending on ADT
Minimum Street Width	26 ft	22-27 ft>2 du 28-34 ft @2-6 du 36 ft< 6 du	16 ft (<100 ADT) 20 ft (100-500 ADT) 26 ft (500-3000 ADT) 32 ft (>6 du/ac)
Additional Right of Way	24 ft	24 ft	8 to 16 ft
Design Speed, Level Terrain	30 mph	30 mph	15 to 25 mph
Curb and Gutter	generally required	generally required	not required on collectors
Cul-de-sac Radii	30 ft	40 ft	30 ft
Turning Radii in Cul-de-sac	20 ft	25 ft	17 ft

Table 2-18. Relationship between number of dwelling units, traffic generation, and residential congestion (Schueler 1995).

No. of Single Family Homes	Average Daily Trips	Peak Trips Per Hour	Minutes between cars (average)	Minutes between cars (peak)
5	50	5	30	12
10	100	10	15	6
25	250	25	6	4
20	500	50	3	1.5
75	750	75	2	45 secs
100	1000	100	1.5	35 secs
150	1500	150	1	20 secs
300	3000	300	30 secs	10 secs

Shoup (1995) and Wilson (1995) summarize the origin of parking “requirements” in urban areas and the overall impact. According to Shoup (1995), motorists report free parking for 99 percent of all automobile trips. About 95% of automobile commuters say that they park free at work. A primary reason for such high use of cars to commute to work is that employers pay for parking. The average for seven case studies of the impact of parking fees on driving behavior is that 72 cars are driven to work per 100 employees if the employer pays for parking while only 53 cars are driven to work per 100 employees if the employee pays for parking (Shoup 1995). Recent state legislation in California requires employers to allow non-auto using employees to receive an equivalent cash payment to the amount of the subsidy for parking.

Between 1975 and 1993, the average number of parking spaces required by cities per 1,000 square feet of office space increased from 3.6 to 3.8 spaces (Shoup 1995). According to Wilson (1995), zoning codes typically require between three and five spaces per 1,000 gross square feet of office building area, with four spaces being the most popular requirement. At 350 square feet per parking space, this corresponds to 1.05 to 1.75 square feet of parking per square foot of office space. Similar ratios have been obtained for residential areas.

The actual estimate of saturation demand for parking is 2.4 spaces per 1,000 square feet of office space for driver paid parking to 3.1 spaces per 1,000 square feet for employer paid parking (Shoup 1995). According to Shoup (1995), over 91% of cities required more than this saturation demand. Wilson (1992) estimated an average requirement of 4.1 spaces per 1,000 square feet in southern California, with the average peak parking demand being only 56% of this capacity.

The primary justification for high parking requirements is to avoid spillover of parking from one parcel of land to others. However, if all facilities are designed for peak demand, often specified as the demand that only occurs 15 to 30 hours per year, then, by definition, large amounts of excess capacity will exist in the system since these peaks are not coincident. According to the Urban Land Institute (1982), specifying a design hour of the 20th busiest hour of the year, leaves spaces vacant more than 99% of the time and leaves half the spaces vacant at least 40% of the time.

Existing parking guidelines have evolved from observing practice around the United States. However, the database is observations on consumer behavior in lots where parking is provided free of charge. Thus, the existing standards are for the demand for parking if parking is free. According to Shoup (1995), virtually no research has been done to determine the optimal amount of parking since parking requirements are usually mandated by the local government agency. If a private developer was free to establish the amount of spaces to provide for his development, the developer would be expected to do a benefit-cost analysis and determine the number of spaces such that his net revenue was maximized.

Many residential streets carry relatively few vehicles each day. For example, streets serving less than 25 homes are so lightly traveled each day (and during peak hours) that shared parking and moving lanes make sense

The requirement for parking is typically estimated from the ITE parking manual (1987). Sample parking requirements are shown in Table 2-19, from Schueler (1995). According to Arnold and Gibbons (1996), the City of Olympia, WA found not only parking oversupply with vacancy rates of 60-70%, but also developers building an average of 51% more spaces than required by the City of Olympia.

Table 2-19. Parking demand ratios for selected land uses and activities (Schueler 1995).

Land Use	Parking Space Ratio Used	Range
Single Family Homes	2 spaces/du	1.5-2.5
Townhouses	2.25 spaces/du	1.5-2.5
Professional Office	1 space/200 sf gfa	150-330
Hotel/Motel	1 space/guest room	0.8-1.25
Retail	1 space/250 sf gfa	200-300
Convenience Store	1 space/300 sf gfa	100-500+es
Shopping Center	1 space/200 sf gfa	150-250
Movie Theatre	1 space/4 seats	3.3-5
Gas Station	2 spaces/pump (and 3 spaces)	
Industrial	1 space/1000 sf gfa	500-1200
Golf Course	4 spaces/hole	3-6.5
Nursing Home	1 space/3 beds	2-4+es
Day Care Center	1 space/8 children	4-10+es
Restaurant	1 space/50 sf gla	0-200
Marina	0.5 space/slip	0.26-0.7+es
Health Club	1 space/100 gfa+es	100-150
Church	1 space/5 seats	4-6
High School	many diverse ratios	
Medical/Dental Office	1 space/175 sf gfa	100-225

Notes: du=dwelling unit, sf=square feet, gla=gross leasable area, es= employee spaces, gfa=gross floor area.

A popular treatment option for parking lots is to deploy street sweepers. Street sweepers are also used for aesthetic purposes. Street sweepers pick up solids and debris. They are much less effective in removing other pollutants. Of course, street sweeping has no impact on the quantity of stormwater runoff. Another potentially effective method is to use porous or permeable pavement to reduce the runoff rates from parking areas.

An important question with regard to parking is the tradeoff between on-street and off-street parking. With contemporary subdivision design, the house has a two or three car garage, a driveway, and parking on the street in front of the house. In some cities, overnight parking on streets is prohibited, thereby increasing the need for off-street parking. A careful reexamination of these policies might show that current neighborhood parking requirements are overly conservative.

Lot Size

Lot sizes and associated dwelling unit densities were discussed previously with regard to estimating imperviousness. Lot size is seen to be a very good overall indicator of the amount of infrastructure needed to support residential development. Trends toward more automobiles and larger houses and a desire for "privacy" have resulted in much larger lot sizes. Demand for larger lot sizes might be reduced if the full costs of these larger lots were assessed on the property owners. In addition to promulgating regulations with regard to right-of-ways, cities often specify lot densities and minimum requirements (Schueler 1995). These minimum setback and related requirements further reduce allowable densities. As with right of ways, it is advisable to revisit these requirements for larger lot sizes.

Dwelling Unit Footprint

Urban dwelling units vary greatly in size as illustrated by these typical units and size ranges:

1. Single room:100-300 sq. ft.
2. Studio apartment: 300-500 sq. ft.
3. One-bedroom unit: 400-700 sq. ft.
4. Two-bedroom unit: 600-1,200 sq. ft.
5. Three-bedroom unit: 1,200-2,500 sq. ft.
6. Four-bedroom unit: 1,800-4,000 sq. ft.

Because of increasing affluence and more affordable housing, the median size of dwelling unit per family has steadily increased since World War II. For example, the median size of home increased from 912 square feet in 1948 to 1,113 square feet in 1963 (ULI 1968, p. 38).

The footprint of the dwelling unit (DU) is the amount of land it occupies. For single story DU's, the sizes of the DU and the footprint are very similar. The footprint is slightly larger due to roof overhang. The footprint is much less than the DU area if multiple level construction is used.

Stormwater runoff from buildings depends upon the roof area and whether the roof downspouts are directly connected to the storm sewer system. At densities of eight or more units per gross acre, the roof area should probably be connected directly to the stormwater control system because insufficient pervious area exists on the property itself. Treatment of roof runoff consists of controlling sources of atmospheric deposition, changing to more benign roofing materials, periodic cleaning of gutters, and disconnecting downspouts. The primary demand management approach is to encourage smaller roof areas by constructing multi-level buildings.

Covered Porches and Patios

The footprint of the DU is increased if covered porches are included in the house. Covered porches are an icon of traditional neighborhood development. One reason that porches fell out of favor is traffic noise. Porches add imperviousness to the property and appear to be regaining popularity. However, porches are a minor source of imperviousness and much of this imperviousness is not directly connected. Thus, no detailed evaluation of porches is included.

Patios may be constructed of permeable or impermeable material. They typically drain to adjacent pervious areas. Also, patios are not a major source of pollutant loadings. Thus, no separate analysis of patios is included.

Garages and Carports

Garages have emerged as an important land use in urban areas during the 20th century. Automobiles require about 200 square feet of garage space per car. As the number of automobiles has continued to increase, so has the number of garage spaces in DU's. Two and three car garages are now the norm for new house construction. The primary runoff from garage areas is from the rooftop. Thus, the impact depends upon whether the roof downspouts are directly connected to the sewer system or discharge to adjacent imperviousness such as driveways.

Treatment of roof runoff consists of controlling sources of atmospheric deposition, changing to more benign materials, and disconnecting downspouts. The primary demand management technique for garages and carports is to reduce the demand for the number of cars. In the United States, there are over 200 million cars for 250 million people. This corresponds to about one vehicle for every licensed driver in the United States. It is possible to have the number of cars per capita continue to increase as people have more than one car per capita.

Driveways

Driveways have become an important source of imperviousness in the 20th century as new developments had to accommodate a growing number of automobiles. The ITE (Southworth and Ben Joseph 1995) recommends minimum driveway widths of 10 feet for one car, with a 20 foot-wide curb cut (five-foot flare on each end). Driveways associated with garages are also an important land use. Four types of driveways need to be considered based on the location and orientation of the garage:

1. Attached, front facing garage
2. Attached, side facing garage
3. Attached, rear facing garage
4. Detached garage in rear of lot

Attached, Front Facing Garage: If the garage faces the street and is attached to the house, then the driveway width is usually the width of the number of garage spaces, or about 9-10 feet of width per car. The length of the driveway depends on the house setback. Minimum driveway lengths are dictated by having sufficient length so that a car can pull into the driveway and not block the sidewalk. Thus, a minimum driveway length is the sum of the distance from the street to the sidewalk (0-15 feet) plus the width of the sidewalk (four-six feet) if there is one plus the length of a car space or about 20 feet, or a total minimum driveway length of 20-41 feet. The extra house setback distance must be added to this minimum distance to get the total distance. For many houses, the paved area for the driveway exceeds the impervious area of the garage. Some, if not all, of the driveway drains to the street, thereby creating a significant source of directly connected impervious area.

Attached, Side or Rear Facing Garage: If the garage entrance faces the side of the house, then a narrower driveway from the street to the house can be used, (e.g., 12 feet). However, this savings in width is offset by the need to provide a turning area so that the cars can maneuver to enter and exit the garage. This added turning area adds significant paved area.

Detached Garage in Rear of Lot: If the garage is detached and located at or near the rear of the lot, then a longer driveway is needed to extend from the street to the rear of the house. The width of this driveway increases in front of the garage to allow cars to enter the various bays. Of course, if an alley exists, then the driveway distance is minimal.

As a low intensity use, driveways are good candidates for porous and permeable pavements or simply paving only parallel strips for the wheels. Another effective control is to route driveway runoff onto adjacent pervious areas instead of directly to the street. This can be done by putting a crown on the driveway as is done for streets.

An effective demand management to reduce the demand for driveways is to reduce the demand for automobiles. Another possibility is to better utilize on-street parking.

Pervious Area on Property

The pervious area on the property is used primarily for lawns, gardens, and wooded areas. This land is used for aesthetic appeal, and recreation for people and pets. Under proposed innovations, this pervious area will be used more intensively to infiltrate stormwater from adjacent impervious areas as well as from precipitation directly onto its surface. At present, pervious areas do receive some of the runoff from impervious areas, primarily from roofs, patios, and some parts of the driveway. Thus, it is important to determine the infiltration capacity of these soils. The infiltration capacity depends on the soil type. Pervious areas can be graded to provide some on-site detention of stormwater, that could then be reused for lawn watering or other purposes. Prince George's County (1997), MD has developed the idea of "functional landscapes" for on-site management of stormwater.

Alleys

Alleys are regaining popularity as part of new urbanism designs. Alleys can be found in older neighborhoods. They provide access for garages and garbage pickup and other deliveries. Alleys eliminate the need for driveways and thereby permit narrower lot widths. Typical alley widths range from 12 to 16 feet. In addition to this pavement width, aprons to the garages on either side of the alley are needed.

Boulder, CO specifies a 20 foot right-of-way width for alleys. The width of the alley is controlled by the required turning radius for vehicles entering and exiting from the garages and open parking areas. From a safety point of view, alleys greatly minimize the traffic and pedestrian safety hazards associated with vehicles entering and backing out of driveways onto the street. Runoff from alleys is directly connected to the storm sewer system. The runoff moves along the alley by overland flow until it reaches the street inlet. Treatment options would be the same as for other impervious areas with low traffic and parking rates. The demand for alleys can be eliminated by using driveways. The tradeoff on the amount of pavement used for alleys vs. driveways depends on the lot geometry.

Sidewalks

Attractive sidewalks are an inducement to walking. According to Chellman (1997), about 10% of Americans walked to work in 1960. By 1990, the percentage walking to work had decreased to 4%. Sidewalks are an integral part of older cities. With lower density urban development, the need for sidewalks is less critical. If the housing density is very low, then people can walk in the street. Also, a single sidewalk can be used instead of having a sidewalk on either side of the street. Sidewalks can be located adjacent to the street or separated by a six to seven foot wide planting area. The ITE (Southworth and Ben Joseph, 1995) recommends sidewalks with a minimum width of five feet on both sides of the street. Sidewalks are typically constructed of reinforced concrete.

The ULI (1968) recommends sidewalks on both sides of the street if the density exceeds six houses per net acre. They recommend five foot wide sidewalks along collector streets and four foot sidewalks on minor streets. Chellman (1997) recommends sidewalk widths of five feet to provide sufficient room for pedestrians to pass without crowding.

Sidewalks typically drain to pervious areas allowing the runoff to infiltrate into the ground. The notable exception is when the sidewalks are located immediately adjacent to the streets; then the sidewalk runoff becomes directly connected since the drainage goes directly onto the streets. A traditional treatment is sweeping the sidewalk areas to keep them clean and to provide trash containers to discourage littering. Sidewalks can be eliminated if the street is safe for non-vehicular use. See the section on streets for a discussion on this topic.

Curb and Gutter and Swales

The curb and gutter serves a number of functions in residential street design including drainage, providing a barrier for vehicles going from the lot to the street or vice versa, and aesthetics. Two primary types of curb and gutter are the barrier curb and the rolling curb. An alternative is to eliminate curb and gutter and allow street runoff to flow onto adjacent pervious areas. The curb and gutter are about two feet in width. The ITE (Southworth and Ben Joseph, 1995) recommends vertical curb with gutters. Rolled curbs are not recommended. However, the ULI (1968) recommends rolled curbs for most residential areas because they avoid curb cuts for driveways.

According to Khisty (1990), curbs are used for the following reasons:

1. Drainage control
2. Pavement-edge delineation
3. Right-of-way reduction
4. Aesthetics
5. Delineation of pedestrian walkways
6. Reduction of maintenance operations

Planting Strip Between Street and Sidewalk

Many subdivision regulations require a planting strip to separate the sidewalk and the street. The ITE (Southworth and Ben Jospeh 1995) recommends planting strips on both sides of the street with a minimum width of six to seven feet and with the planting strip draining towards the street. A 1990 revision of these standards decreased the minimum planting width to five feet. Boulder, CO specifies an eight foot wide planting area. Planting strips with a width of 15 feet are popular in the western suburbs of Chicago. These planting strips provide a buffer between the street and sidewalk. They also provide a planting area within the right of way for trees. Early subdivision regulations promulgated by the federal government suggested two trees should be planted on each lot. Drainage from these planting areas is directed towards the street. No citations could be found regarding how these areas could function as part of the stormwater drainage system. They could be expected to attenuate noise and air pollution effects to a limited degree.

Overall Right of Way

Required right of way width dimensions for Boulder, CO are (Boulder 1982):

1. Bikeway: 12 ft
2. Alley: 20 ft
3. Residential: 48 ft
4. Residential collector: 68 ft
5. Collector: 81 ft
6. Arterial: 130 ft
7. Freeway: Use AASHTO standards

To this base are added medians, added travel lanes and speed changing lanes, and turn lanes. These right-of-way requirements are typical. The key control option is to take a hard look at existing right-of-way requirements, especially in residential areas, to see whether the requirements could be modified to reduce the generation of impervious area that is providing little or no added value and to encourage the more effective use of pervious areas within the right-of-way.

Will Americans Reduce Auto Use?

Dittmar (1995) outlines a broader context for transportation planning that incorporates some of the above concepts for developing more sustainable transportation systems. In his conclusions, he discusses the feasibility of reversing the trend since World War II of increasing reliance on the automobile. Dittmar says:

> ***In discussions of the issues with transportation officials, their most frequent initial assertion is that Americans love cars and cherish driving, and that any reform effort is therefore somehow doomed. Running a close second are the assertions that Americans are voting with their gas pedals by choosing exurbia, and that building more roadways is simply giving folks what they want. I don't believe this is true. People are responding to a set of signals our society gives them by building ring roads and beltways, subsidizing free parking and suburban development through utility infrastructure, and providing tax incentives that favor car use and suburban home ownership. These signals favor continued sprawl and reliance on cars. Changing these endemic signals by creating incentives to live in the city, eliminating tax biases toward cars, and enhancing livability can send the public new signals.***

With regard to streets, parking, and other major sources of imperviousness, engineers have been the ones who have promulgated these regulations. Hopefully, they can also take the lead in modifying them to create more sustainable communities.

Summary and Conclusions

The results of this discussion on the nature of imperviousness in urban areas show that the quantity of urban stormwater generated per dwelling unit has increased dramatically during the 20th century due to the trend towards more automobiles which require more streets and parking, and the trend towards larger houses, all combined on larger lots. Commercial and industrial areas likewise need much more parking per unit of office space than they did before automobiles. Interestingly, the square footage for residential and commercial areas is less than the support parking requirements. Modern practices dictate devoting more of the city landscape to parking than to human habitat and commercial activities. The net result of this major shift in urban land use is low density sprawl development that generates over three times as much stormwater runoff per family than did pre-automobile land use patterns. Much of these requirements for more and wider streets and parking have been mandated in order to improve the transportation system. Ironically, unlike water infrastructure, these services are not charged directly to the users. Rather, they are subsidized by the general public including non-users. Options for changing this pattern are presented in Chapter 3.

References

AASHTO. (1984). A Policy on Geometric Design of Highways and Streets. Washington, D.C.

Akbari, H., S. Davis, S. Sorsano, J. Huang, and S. Winnett Eds. (1992). Cooling Our Communities-A Guidebook on Tree Planting and Light-Colored Surfacing. U.S. Environmental Protection Agency, Washington, D.C., 217 p.

Amekudzi, A.A., N.O. Attohokine, and S. Laha (1997). Brownfields Redevelopment Issues At The Federal, State, And Local Levels. J. Environmental Systems, Vol. 25(2), p. 97-121.

American Public Works Association (1968). The Causes and Remedies of Water Pollution From Surface Drainage in Urban Areas-Research Project No. 120. Final Report to Federal Water Pollution Control Administration-WP-20-15, Washington, D.C.

Arnold, Jr., C.L. and C.J. Gibbons (1996). Impervious Surface Cover-The Emergence Of A Key Environmental Indicator. Jour. of the American Planning Assoc., Vol. 62, No.2, p. 242-258.

Barnett, J. (1992). Sustainable Development: How To Make It Work. Architectural Record, June, p. 32.

Barnette, C.H. (1995). Revitalizing Brownfield Sites. Iron Age New Steel, 1:6, p. 88.

Ben-Joseph, E. (1995). Changing The Residential Street Scene. Journel of the American Planning Association. Vol. 61, No.4, autumn. P.504-515

Binkley, C., B. Collins, L. Kanter, M. Alford, M. Shapiro and R. Tabors (1975). Interceptor Sewers and Urban Sprawl. Lexington Books, Lexington, MA. 118 p.

Bookout, L.W. (1992). Neotraditional Town Planning-The Test Of The Marketplace. Urban Land, Vol. 51, June, p. 12-17.

Bookout, L.W. (1994). Valuing Landscape, Site Planning, And Amenities. Urban Land, Vol. 53, Nov., p. 39-43.

Booth, D. and L. Reinelt (1993). Consequences Of Urbanization On Aquatic Systems-Measured Effects, Degradation Thresholds, And Corrective Strategies. Proc. Watershed Management. WEF, Alexandria, VA.

Boulder, CO. (1982). Design Criteria and Standard Specifications. 11/9.

Boulder, CO. (1993). 5/26/93 Memo On Downtown Parking.

Calthorpe, P. (1993). The Next American Metropolis-Ecology, Community and the American Dream. Princeton Architectural Press.

CH2M Hill (1993). Cost of Providing Government Services to Alternative Residential Patterns. Report for the Chesapeake Bay Program's Subcommittee on Population Growth and Development. USEPA, Contract No. 68W0-0043.

Chellman, C.E. (1997). Traditional Neighborhood Development Street Design Guidelines. Institute of Transportation Engineers. Washington, D.C., 43 p.

Chesapeake Bay Foundation (1996). Growth, Sprawl and the Bay-Simple Facts About Growth and Land Use. Chesapeake Bay Foundation. September.

Clark, R. (1983). Economics Of Regionalization: An Overview. ASCE. New York, NY.

Colatins, E. and C. Bartsch (1996). Industrial Site Reuse And Urban Redevelopment-An Overview. Cityscape: A Jour. of Policy Development and Research. Vol. 2, No. 3, p. 17-61.

Deakin, E. (1995). Land Use and Transportation Planning in Response to Congestion Problems: A Review and Critique. Transportation Research Record 1237. p. 77-86.

Debo, T.N. and A.J. Reese (1995). Municipal Storm Water Management. Lewis Publishers. Boca Raton, FL. 756 p.

Dittmar, H. (1995). A Broader Context for Transportation Planning. Jour. of the American Planning Assoc. Vol. 61, No. 1, p. 7-13.

Engel, D., E. Stromberg and M.A. Turner (1996) Toward A National Urban Environmental Policy. Cityscape: A Jour. of Policy Development and Research, Vol. 2, No. 3, p. 1-16.

Ewing, R. (1994). Characteristics, Causes And Effects Of Urban Sprawl: A Literature Review. Environmental and Urban Issues. Vol. 21, Winter, p. 1-15.

Ewing, R. (1996). Best Development Practices. Planners Press. American Planning Association. Chicago, IL.

Ewing, R. (1998). Best Development Practices-A Primer. USEPA. Washington, D.C.

Ferguson, B.K. (1994). Stormwater Infiltration. Lewis Publishers. Boca Raton, FL. 269 p.

FHWA. (1997). Ultra-urban BMPs. Draft.

FHWA. Undated. Is Highway Runoff a Serious Problem? www.tthrc.gov/hnr20/runoff/runoff.html.

Forbes, T.W. (1981). Human Factors in Highway Traffic Safety Research. Malabar, FL. Robert G. Kreiger Publishing Co.

Geltman, R.B. (1997). Brownfield Financing Strategies. Environmental Regulation and Permitting. Spring, p. 5-7.

Goldstein, H (1997). A City of Dreams. Colorado Commons. Spring, P.42-44

Graham, Jr., E. (1944). Natural Principles of Land Use. Greenwood Press. New York., NY.

Hanousek, D. et al. (1996). Project Infrastructure Development Handbook. ULI. Washington, D.C. 178 p.

Hanson, M. (1992). Automobile Subsidies And Land Use. Jour. of the American Planning Assoc. 58:1.

Harza Engineering Company and Bauer Engineering Company, Incorporated (1966). Flood and Pollution Control: A Deep Tunnel Plan for the Chicago Metropolitan Area.

Hawken, P. (1993). Ecology and Commerce. Harper Collins. New York, NY.

Heaney, J., W. Huber, and S. Nix (1977). Storm Water Management Model: Level I-Preliminary Screening Procedures. U.S. Environmental Protection Agency. EPA – 600/2-76-275. Cincinnati, OH.

Howard, E. (1965). Garden Cities of Tomorrow. Reprint, F.J. Osborn, Ed. MIT Press. Cambridge, MA. Originally published in 1878.

Huang, Y.J., H. Akbari and H.G. Taha (1990). The Wind-Shielding And Shading Effects Of Trees On Residential Heating And Cooling Requirements. 1990 ASHRAE Trans. (Atlanta, GA). Also Livermore Berkeley Lab Report 24131.

ICF Incorporated and Apogee Research Inc. (1997). Opportunities to Improve Air Quality through Transportation Pricing Programs. US Environmental Protection Agency. EPA-420-R-97-004. Washington, D.C.

Institute of Transportation Engineers (1965-90). Traffic Engineers Handbook. ITE. Washington, D.C.

Institute of Transportation Engineers (1987). Parking Generation. 2nd Ed. ITE. Washington, D.C.

Institute of Transportation Engineers (1989). Residential Street Design and Traffic Control. Prentice-Hall. Englewood Cliffs, NJ.

James, W. Ed. Current Practices in Modelling the Management of Stormwater Impacts. Lewis Press. Boca Raton, FL.

Kandelaars, P., H. Opschoor, and J. van den Bergh (1996). A Dynamic Simulation Model For Materials-Product-Chains: An Application To Gutters. J. Environmental Systems. Vol. 24(4), p. 345-371.

Katz, B. (1997). Brookings National Issues Forum on Metropolitan Solutions to Urban and Regional Problems. www.smartgrowth.org/library/katz.html.

Katz, P. (1994). The New Urbanism-Toward and Architecture of Community. McGraw-Hill. New York, NY.

Khisty, C.J. (1990). Transportation Engineering. Prentice-Hall, Inc. Englewood Cliffs, NJ.

Knapp, G. (1998). The Determinants of Residential Property Values: Implications for Metropolitan Planning. Jour. of Planning Literature. 12, 3.

Kunstler, J. H. (1996). Home From Nowhere. Simone and Schuster, New York, NY

Lee, Jr., D.B. (1973) Requiem For Large-Scale Models. Journal of the American Institute of Planners. Vol. 39, p.163-178.

Limpert, R. (1994). Motor Vehicle Accident Restoration and Cause Analysis. 4th Ed. Charlottesville, VA. The Michie Co.

Littman, T. (1998). Transportation Cost Analysis: Techniques, Estimates, and Implications. Victoria Transport Policy Institute. Victoria, BC.

Marsh, G.P. (1970). The Earth as Modified by Human Action, Arno Press (originally published in 1874 as Man and Nature.

McHarg, I. (1971). Design with Nature. Doubleday. Garden City, NY.

Mohney, D. and Easterling, K., Editors (1991: Seaside; Making a Town in America. Princeton University Architectural Press, New York, New York

Moss, M.L. (1997). Reinventing The Central City As A Place To Live And Work. Housing Policy Debate. Vol. 8, Issue 2, p. 471-490.

National Commission on the Environment (1993). Choosing a Sustainable Future. p. 114. Nation. Island Press. Washington, DC. 180 pages.

National Safety Council (1995). Accident Facts. Itasch, IL

Newsweek (1995). 15 Ways to Fix the Suburbs. May 15

Novotny, V. and H. Olem (1994). Water Quality-Prevention, Identification, and Management of Diffuse Pollution. Van Nostrand Reinhold. New York, NY. 1054 p.

O'Connor, K.A. (1996). Institutional Framework For Water Quality Over The Past Century. in WEF Watershed 96. Alexandria, VA. p. 1019.

Oglesby, C.H. and R.G. Hicks (1982). Highway Engineering. 5th Ed. J. Wiley and Sons. New York, NY.

Oknn, D.A. (1991) Reclaimed Water: An Urban Resource. Water Science and Technology. Vol. 24, No. 9, p. 352-362.

Papacostas, C.S. and Prevedouros (1987). Transportation Engineering and Planning. 2nd Ed. Prentice-Hall. Englewood Cliffs, NJ.

Parente, M. and M.E. Hulley (1994). Stormwater Pollution at a Major Highway Interchange. Chapter 10 in (missing ?????)

Peirce, Neil (1994). The Dawn Of Civic Environmentalism. Tampa Tribune. Jan. 10.

Pitt, R. and Voorhees, J. (1994). Source Loading and Management Model (SLAMM). Department of Civil and Environmental Engineering. University of Alabama at Birmingham.

Prince George's County (1997). Low-Impact Development Design Manual. Prince George's County, MD.

Real Estate Research Corporation (1974). The Cost of Sprawl. U.S. Government Printing Office. Washington, D.C.

Renner, M. (1988). Rethinking The Role Of The Automobile. World Watch Institute. Washington, D.C.

Repogle, M. (1991). Sustainability: A Vital Concept For Transportation Planning And Management. Jour. of Advanced Transportation. 25, No. 1, p. 9.

Risse, E.M. (1989). The American Settlement Pattern of the 21st Century-Where are the "Sub"urbs Going. www.smartgrowth.org/library/risse/html.

Roesner, L.A., et al. (1991). Hydrology of Urban Runoff Quality Management. Proc. 18th National Conf Water Resources Planning and Management. Symposium Urban Water Resources. American Society of Civil Engineers. New York, NY.

Schueler, T. et al. (1992). Watershed Restoration Sourcebook. Metro. Washington Council of Governments. Washington, D.C.

Schueler, T.R. (1994). Site Planning For Urban Stream Protection. Watershed Protection Techniques. 1,3: 137-140.

Schueler, T.R. (1995). Site Planning for Urban Stream Protection. Metro. Washington Council of Governments. Washington, D.C. 232 p.

Shoup, D.C. (1995). An Opportunity To Reduce Minimum Parking Requirements. Jour. American Planning Assoc. Vol. 61, No. 1, p. 14-28.

Smith, W. (1964). The Low-Rise Speculative Apartment. Research Report 25. Berkeley: University of California Center for Real Estate and Urban Economics.

So, F.S. and J. Getzels, Eds. (1987). The Practice of Local Government Planning. 2nd Edition. International City Managers Association. Washington, DC

Southworth, M. and E. Ben-Joseph (1995). Street standards and the shaping of suburbia. American Planning Assoc. Journal. Vol. 61, No. I, p. 65-81.

Southworth, M. and E. Ben-Joseph (1997). Streets and the Shaping of Towns and Cities. McGraw-Hill. New York, NY.

Sunset (1992). Best Home Plans. Sunset Publishing Co. Menlo Park, CA.

Tetra Tech (1996). Green Development Literature Search: Summary and Benefits Associated with Alternative Development Approaches. USEPA. Washington, D.C. www.smartgrowth.org/bibliographies/greenlit_search/case_examples.html.

Thomson, N.R., E.A. McBean, I.B. Mostrenko, and W.J. Snodgrass (1994). Characterization of Stormwater Runoff from Highways. Chapter 9 in James, W. Ed. Current Practices in Modelling the Management of Stormwater Impacts. Lewis Press. Boca Raton, FL.

Urban Land Institute (1968). The Community Builders Handbook. Urban Land Institute. Washington, D.C. 526 p.

Urban Land Institute (1989). Project Infrastructure Development Handbook. ULI. Washington, D.C. 175 p.

Urban Land Institute (1982). Parking Requirements for Shopping Centers. Washington, D.C. Urban Land Institute.

Urbonas et al. (1990). Optimization Of Stormwater Quality Capture Volume. In Urban Stormwater Quality Enhancement-Source Control. Retrofitting and Combined Sewer Technology. ASCE. New York, NY.

USEPA. (1991). Proposed Guidance Specifying Management Measures for Sources of Nonpoint Pollution in Coastal Areas. May.

Wagener, M. (1994). Operational Urban Models: State Of The Art. Journal of the American Planning Association. Vol. 60, No. 1, p.17-19.

Walesh, S.G. (1989). Urban Surface Water Management. Wiley and Sons. New York, NY.

Walesh, S.G., and R. Carr (1998). Controlling Stormwater Close To The Source: An Implementation Case Study. American Public Works Congress. Las Vegas, NV. September.

Walter, B., L. Arkin and R. Crenshaw (1992). Sustainable Cities Concepts And Strategies For Ecocity Development. Ecohome Media. p. 178.

Water Environment Federation-American Society of Civil Engineers (1998). Urban Runoff Quality Management. WEF Manual of Practice N. 23. ASCE Manual and Report on Engineering Practice No. 87. Alexandria, VA and Reston, VA. 259 p.

Wilson, R. (1992). Suburban Parking Economics and Policy: Case Studies of Office Worksites in Southern California. Report No. FTA-CA-11-0036092-1. Washington, D.C. U.S. Dept. of Transportation.

Wilson, R.W. (1995). Suburban Parking Requirements: A Tacit Policy For Automobile Use And Sprawl. Jour. of the American Planning Assoc. 61, 1: 29-42.

World Commission on Environment and Development (1987). Our Common Future

Wright, P.H. and R.J. Paquette (1996). Highway Engineering. J. Wiley and Sons. New York, NY.

Young, G. K., Stein, S., Cole, P., Kammer, T., Graziano, F., and Bank, G. (1966). Evluation and Management of Highway Runoff Quality. Publication No. FHWA-PD-96-

032. Federal Highway Administration. Washington, DC, p.480.

Huhnke, B. (1997). Take a Deep Breath. Colorado Commons. Spring pp. 20-21.

Zimmerman, R. and Sparrow, R. (1997). Workshop in Integrated Research for Civil Infrastructure. Final report to NSF, Washington, DC

Chapter 3

Sustainable Urban Water Management

James P. Heaney, Len Wright, and David Sample

Introduction

Water supply, wastewater, and stormwater systems are explored in this chapter, first individually and then looking at them in an integrative manner. Key areas of potential integration of these three functions are in reuse of wastewater and stormwater to reduce the required net import of water for water supply. The literature review summarizes previous and on-going work nationally and internationally to develop more sustainable urban water management systems.

Systems View of Urban Water Management

The mid 1960's were a period of great change in the water resource field in the United States. The 1964 Water Resources Research Act established the Office of Water Resources Research (OWRR) with a mission of promoting interdisciplinary research because the individual federal agencies were only looking at their mandated piece of the total water system. Also, the 1965 Water Resources Planning Act established river basin commissions to better integrate water resources planning across federal agencies. Great strides were made in urban water and environmental management during the 1960's and 1970's because of strong federal support for research, a national mood to look at revitalizing our cities and restoring the environment, and the concomitant emergence of the systems approach and essential computer hardware and software.

The leadership in urban water resources during the early years can be traced to the ASCE Urban Water Resources Research Council (UWRRC) headed by M.B. McPherson. With funding from OWRR and the National Science Foundation (NSF), the UWRRC sponsored research conferences and numerous research projects dealing with a wide variety of urban water resources issues. The early results are published in McPherson et al. (1968). They pointed out that:

> **A single aspect research approach is totally inadequate and, indeed, is entirely inappropriate, for resolving multi-aspect problems. The former simplistic approach of regarding a unit of water as a fixed entity, such as stormwater, must be abandoned for that same unit at a different point in time will be categorized as water supply, recreation, esthetics, etc., perhaps several times before leaving a given metropolis.**

The ASCE UWRRC defined urban water resources to consist of:

1. Urban water uses:
   - Water supply (domestic, commercial, agricultural and for fire protection).
   - Conveyance of wastes (from buildings and industries).
   - Dilution of combined and storm sewerage system effluents and treatment plant effluents (by receiving bodies of water).
   - Water-oriented recreation and fish management.
   - Aesthetics (such as landscaped creeks and ponds in parks and parkways).
   - Transportation (commercial and recreational).
   - Power generation.

2. Protection of urban areas from flooding:
   - Removal of surface water at the source.
   - Conveyance of upstream surface water through the area.
   - Barricading banks, detaining or expressing flow natural streams to mitigate spillover in occupied zones of flood plain.
   - Flood proofing of structures.

3. Manipulation of urban water:
   - Groundwater recharge.
   - Recycling of water.

4. Pollution abatement in urban areas:
   - Conveyance of sanitary sewage and industrial wastes in separate sewerage systems.
   - Interception of sanitary sewage and industrial wastes.
   - Interception and treatment of storm sewer discharges or combined sewer overflows.
   - Reinforcing waste assimilative capacity of receiving water bodies.
   - Treatment of sanitary wastes at point of origin.

5. Interfacial public services:
   - Snowstorm and rainstorm traffic routing.
   - Street cleaning scheduling.
   - Snow removal strategies.
   - Lawn irrigation conservation.
   - Air pollution control.

The review of the integrated approach to urban water systems, which was in vogue in the late 1960's and 1970's, indicates that these researchers had scoped the problem very well. The spatial scale for these early systems studies tended to be macro in that it encompassed the entire urban area with a view towards finding the most cost-effective overall system. This approach was compatible with federal infrastructure funding patterns that required that the funded projects be part of an overall transportation or wastewater master plan for the entire urban area.

A systems approach to urban water management was described by Jones in 1971 (see Figure 3-1). McPherson (1973) argued that developing an urban water budget was an essential first step in using a systems approach as shown in Figure 3-2. Concurrently, researchers at Resources for the Future were stressing the use of a materials balance approach for inventorying and evaluating the generation and disposal of "residuals" or the quality constituents associated with transport in the air or water (Kneese, Ayres, and d'Arge 1970). A more recent summary of the residual management approach and a comprehensive catalog of models is presented in Basta and Bower (1982). Heaney (1994) presents an overview of these early studies.

Sustainability Principles for Urban Water Infrastructure

With regard to urban development in general and urban water systems in particular, Grottker and Otterpohl (1996) list the following general principles for providing sustainable development:

For the same or more activities, use less energy and material.

- Do not transfer problems in space or time to other persons.
- Minimize degradation of air, water, and land.

Application of these principles to urban water systems yields the following principles (Grottker and Otterpohl 1996):

1. Minimize the distance of water and wastewater transportation.
2. Use stormwater from roofs, preferably for water supply, instead of infiltrating or
   discharging it.
3. Do not mix the human food cycle with the water cycle. Do not mix waste waters
   of different origin.
4. Decentralize urban water systems and do not allow human activities with water if
   local integration into the water cycle is not possible.
5. Increase the responsibility of individual humans for their impacts on local water and wastewater systems.

THE SYSTEMS APPROACH TO URBAN WATER RESOURCES

THE URBAN COMPLEX IS THE BASIC SYSTEM:

The urban complex is people and serves people.

THE URBAN WATER RESOURCE IS A SUBSYSTEM IN THE BASIC URBAN SYSTEM:

To address the urban water resource as an independent system, even for convenience, may lead to dangerously narrow conclusions.

TRADITIONAL THINKING OF WATER SUPPLY, DISTRIBUTION, SEWAGE, FLOOD CONTROL, AND RECREATION AS SUB-ORDERS MAY BE INAPPROPRIATE:

These are interdependent service functions.
Perhaps the following breakdown might prove better:
The complete water cycle.
The environment, including people.
The ecology, including people (if separable from environment).
Public and private economies.
Management.

GENERALIZATIONS AT THE SUB-SUB-SUBSYSTEMS LEVEL COULD DEFEAT THE OBJECTIVES OF THE SYSTEMS APPROACH:

The progress of science is measured by development of details. Research contributions typically come from multiple minute steps--not from giant strides forward.
Rewarding concepts, innovations and improvements will originate essentially at the sub-sub-subsystem level.

TEMPTATIONS TO GENERALIZE, TO INERRELATE ONLY WITHIN THE FINITE CAPABILITY OF A MACHINE, AND TO IGNORE "INTANGIBLE" RELATIONSHIPS LACKING HARD DATA, MUST BE AVOIDED:

Neither a model nor a machine can think.
Man cannot excuse his failure to think.

Figure 3-1. Early view of the systems approach to urban water management (Jones 1971).

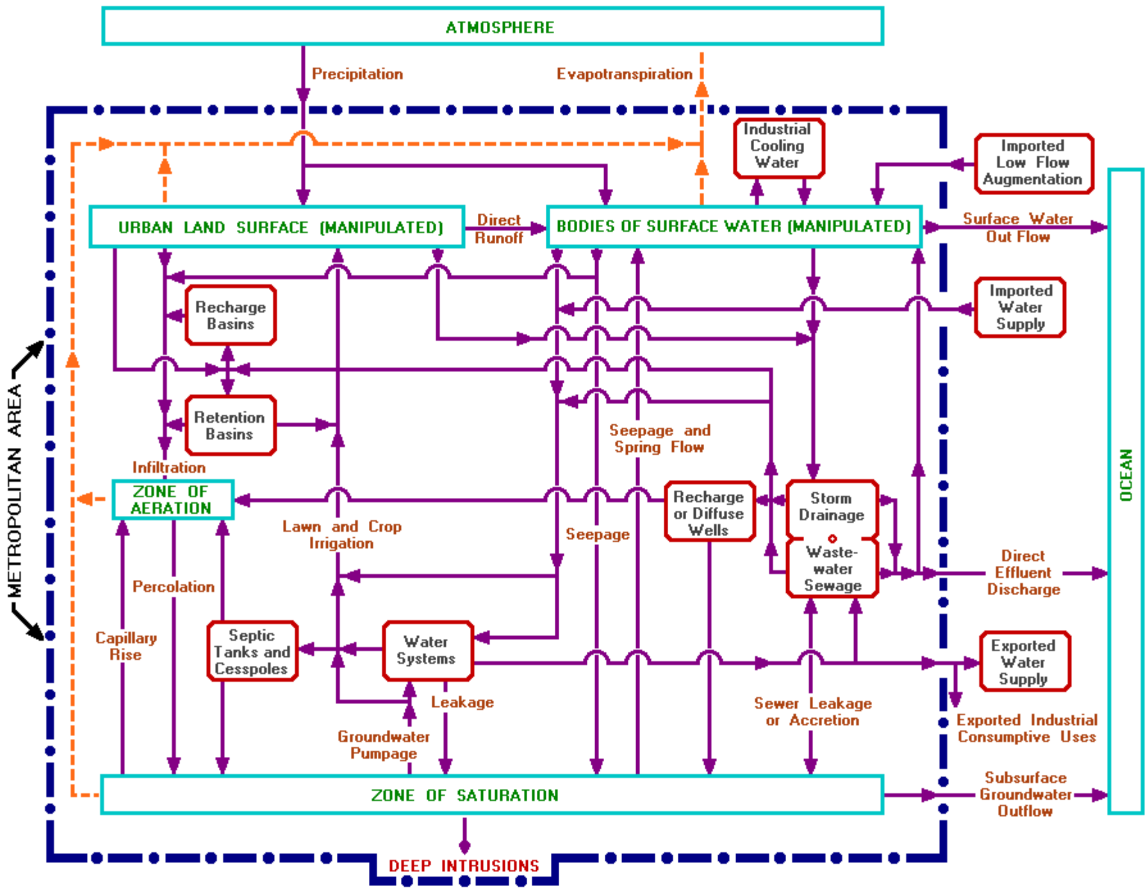


Figure 3-2. Water budget for urban water systems (McPherson 1973).

Sustainability has become popular as a general goal of future societies in general and environmental and economic systems in particular. A recent issue of Water Science and Technology featured numerous articles by European authors on the theme of “Sustainable Sanitation” (Henze et al. 1997). They could not find an operational definition of sustainability as it applies to urban water problems. Several authors did strongly advocate taking an holistic view of urban water systems ranging from water supply to wastewater and stormwater collection, treatment, and disposal.

Clark, Perkins, and Wood (1997) have developed and applied concepts of sustainability to evaluating alternative futures for the water system in Adelaide, Australia. This effort is the largest known case study of a group that is taking an integrative look at this problem. The purpose of the Water Sustainability in Urban Areas (WSIUA) project is to investigate the feasibility and benefits of progressive replacement of the existing large scale, single purpose water systems with replicated small scale, multipurpose water systems. These water systems consist of water supply, wastewater and stormwater. The key concepts explored in this study are

(Clark et al. 1997):

1. Adoption of a long planning cycle compatible with the life span of major components of the water systems.
2. Planning water systems to achieve multiple objectives-environmental, social, and economic.
3. Viewing water as a valuable resource warranting conservation and efficient utilization.
4. Undertaking water planning which seeks efficiency gains through taking a total water cycle approach on a local and regional basis as the best means of meeting multiple objectives.
5. Integrating water systems as appropriate to achieve efficiencies through infrastructure cost sharing.
6. Localizing water systems to achieve efficiencies through maximizing local opportunities.
7. Utilizing rainwater capture, effluent recycling and groundwater storage to maximize system resilience.
8. Franchising the operation of small scale systems as the best means of balancing cost competition with maintenance of adequate reliability and public health standards.
9. Recognizing the organizational and social implications of integrated local water systems.

Urban Water Budget

Literature Review

Water budgets have become popular in recent years as water professionals attempt to do more holistic evaluations of urban water systems. Grimmond et al. (1986) present a schematic of the components of the urban water budget as shown in Figure 3-4.

Stephenson (1996) cites three impacts of urbanization on stormwater runoff:

- Increased stormwater runoff.
- Recession of the water table.
- Shorter response time due to imperviousness.

He compares the water budgets of an undeveloped catchment with an urbanized catchment in Johannesburg, South Africa. The results show the expected increase in direct runoff and the need to import water for water supply. He also cites an urban water budget of a suburb of Vancouver, B.C. (Grimmond and Oke 1986).

Nelen et al. (1996) describe the planning of a new development for about 10,000 people in Ede, Netherlands. The three underlying environmental principles are sustainability, quality, and ecology. This area has a high groundwater table so groundwater management is an important part of the project. They plan to incorporate water-conserving hardware and divert the more polluted stormwater into the sanitary sewer. In addition, they are considering a dual water supply system.

Fujita (1996) describes efforts in Japan to encourage stormwater infiltration. The multiple objectives of this approach include:

1. River flow maintenance.
2. Springwater restoration.
3. Water resources guarantee.
4. Ground subsidence prevention.
5. Groundwater salination prevention.

Herrmann and Klaus (1996) do general water and nutrient budgets for urban water systems including stormwater. Imbe et al. (1996) performs a water budget analysis to determine the impact of urbanization on the hydrological cycle of a new development near Tokyo, Japan. This development is trying to minimize hydrologic impacts by encouraging infiltration systems and storing rainwater. Mitchell et al. (1996) describes a water budget approach to integrated water management in Australia. Budgeting is done at the individual parcel, neighborhood, and wider catchment scale. On-site management options include providing rain and graywater storage.

Clark et al. (1997) uses a water budget approach to evaluate decentralized urban water infrastructure for Adelaide, Australia.

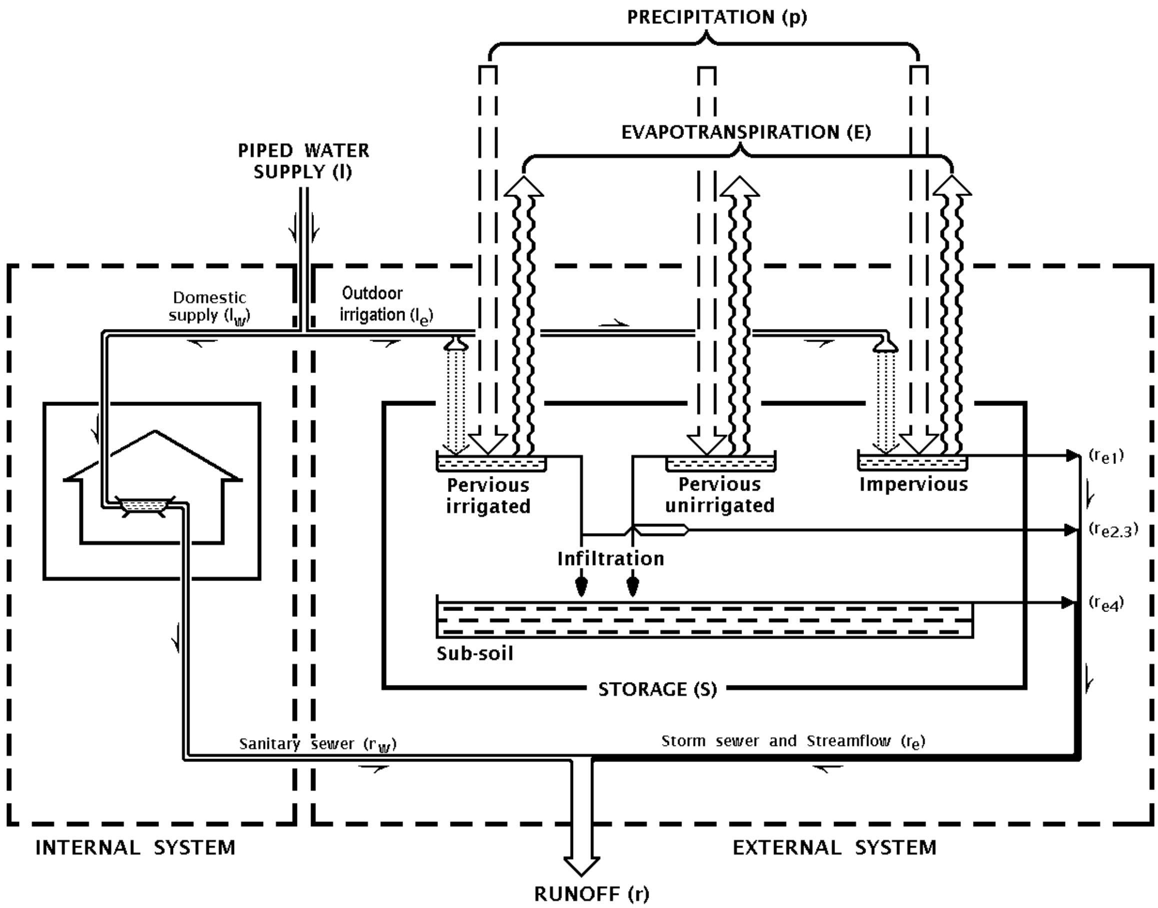


Figure 3-3. The urban hydrologic system (Grimmond et al. 1986).

Dry Weather Urban Water Budget

Urban water use, wastewater, and urban stormwater are interdependent. Virtually all of the indoor water use is discharged to separate or combined sewers. The total quantity of wastewater is strongly influenced by infiltration and inflow, which often increase as a result of wet-weather conditions. Outdoor urban use for irrigation of plants makes pervious areas wetter and reduces the potential soil moisture storage available during wet weather periods. However, properly managed, a significant portion of urban stormwater can be directed onto pervious areas to reduce irrigation needs. These interactions and the potential for better integration of uses are described in this section.

The residential water demand data presented in this study are based on the results of a national study sponsored by the American Water Works Association Research Foundation (AWWARF) and 12 participating cities. Using an innovative monitoring and data logging system, detailed water use was monitored for approximately 1,200 houses in 12 cities. Each house was monitored for two weeks in a warmer period and 12 weeks in a colder period. Readings were taken every 10 seconds and converted into individual water using events using specially developed software. This work was finished in April 1998. This project is referred to as the North American End Use Study (NAREUS) project. Descriptions of this effort can be found in DeOreo et al. (1996), Harpring (1997), Mayer et al. (1997), Stadjuhar (1997), or by visiting the homepage of Aquacraft at www.aquacraft.com. The summary results of this water use study are presented in Table 3-1 that describes overall water use in the 12 cities, and Tables 3-2 and 3-3 that present the city summaries for each sampling period so that the reader can see the difference between the results for the warmer versus the colder periods.

Indoor Urban Residential Water Use

The results of the NAREUS project indicate an average indoor water use of 63.2 gallons per capita per day (gpcd) with a range from 49 to 73 as shown in Table 3-1. Perusal of Tables 3-2 and 3-3 indicates that indoor water use does not vary significantly between winter and summer. Indoor residential water use per capita is quite stable in the United States reflecting the fact that indoor water use is for relatively essential purposes. These results are quite similar to previous studies of indoor water use. Based on a nationwide evaluation, Maddaus (1987) concluded that indoor residential water use averaged 60 gpcd. Studies of the expected value of wastewater into sewers likewise report an average of 60 gpcd. Toilets account for the largest percentage of indoor water use in all three studies followed by clotheswashers, showers, and faucets. The basis for the results shown in these three studies is described below.

Indoor water use does not vary significantly over the year. Some daily variability occurs between weekdays and weekends. The hourly distribution of indoor residential water use is shown in Figure 3-5 (Harpring 1997). Peak usage occurs during the early morning hours of 7 am to 10 am. Most of this peak is due to toilet and shower use. Toilet flushing continues at a similar rate for the rest of the day and into the evening. On the other hand, showers are taken primarily in the morning. Peak clothes washing activity occurs from 9 am to 1 pm. In general, water use in houses declines during the middle of the day because fewer people are at home. Use increases in the evening as people return home and prepare dinner, and then reaches its lowest level between midnight and 6 am when people are asleep. Interestingly, the British studies show use during the early morning hours for dish and clothes washing. The explanation for this usage pattern is that customers are taking advantage of lower electric rates during these hours (Edwards and Martin 1995). A general discussion of expected future trends in indoor water use follows.

Table 3-1. Summary of indoor water use for 12 cities in North America

All values in gallons per capita per day

	2.42	2.74	2.46	2.80	2.73	2.42	2.86	2.34	3.12	3.28	3.07	2.81	2.75	0.30	0.11	
	Boulder	Denver	Eugene	Seattle	San Diego	Tampa	Phoenix	Scottsdale/	Waterloo	Walunt Valley	Las Virgenes	Lompoc	Average of	Std. Dev. Of	Coef. Of	
User Category	Colorado	Colorado	Oregon	Washington	California	Florida	Arizona	Tempe, AZ	Ontario	California	California	California	12 cities	12 cities	Variation	% of total
Baths	1.58	1.51	1.29	0.66	0.57	1.12	0.95	0.95	1.35	1.05	1.28	1.41	1.16	0.30	0.26	1.83%
Clothes Washers	13.76	14.65	16.02	10.82	15.77	13.49	14.80	13.70	12.70	13.90	16.30	15.75	14.30	1.59	0.11	22.64%
Coolers	0.16	0.34	0.00	0.00	0.00	0.10	1.50	2.45	0.00	0.02	0.05	0.00	0.38	0.78	2.02	0.61%
Dish Washers	1.42	1.07	1.26	0.82	0.83	0.60	0.75	1.05	0.75	0.60	0.87	0.82	0.90	0.25	0.28	1.43%
Faucets	10.47	8.86	9.70	6.86	9.89	10.28	8.28	9.63	9.53	10.51	9.91	7.84	9.31	1.13	0.12	14.74%
Drinking water*	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	-	-	0.58%
Hot Tubs	0.02	0.00	0.00	0.00	0.10	0.00	0.05	0.20	0.00	0.30	0.26	0.05	0.08	0.11	1.34	0.13%
Leaks**	3.38	6.62	8.70	4.14	4.56	9.75	12.45	14.80	6.20	7.35	9.25	8.46	7.97	3.37	0.42	12.62%
Showers	13.17	12.73	13.73	10.10	9.14	9.92	11.90	11.75	8.15	11.60	11.06	11.46	11.22	1.65	0.15	17.76%
Toilets	20.66	17.96	20.29	14.64	15.02	15.16	17.00	17.05	17.50	16.10	14.22	14.59	16.68	2.16	0.13	26.40%
INDOOR	65.02	64.48	71.41	48.62	56.23	60.78	68.65	72.70	61.80	63.15	63.94	61.48	63.19	6.50	0.10	100.00%
Outdoor	78.40	115.59	82.59	38.19	66.57	33.75	108.50	163.60	13.40	86.55	168.76	37.81	82.81	49.72	0.60	
TOTAL	143.42	180.07	154.00	86.81	122.79	94.53	177.15	236.30	75.20	149.70	232.70	99.28	146.00	53.74	0.37	

*Drinking water at 1.4 liters per capita per day

**Leaks are assumed to be indoor. They actually are a combination of indoor and outdoor.

***Unknown is assumed to be outdoor. It is actually a combination of indoor and outdoor.

Table 3-2. Summary of indoor and outdoor water use in Boulder, Denver, Eugene, Seattle, and San Diego

GALLONS PER CAPITA PER DAY

	BOULDER, CO			DENVER, CO			EUGENE, OR			SEATTLE, WA			SAN DIEGO, CA			TAMPA, FL			Six City
	5/21 - 6/6/96	9/1 - 9/19/96	Average	6/5 - 6/21/96	10/29-11/14/96	Average	6/25-7/11/96	12/2-12/20/96	Average	7/17-8/1/96	1/8-1/24/97	Average	8/7-8/25/96	2/5-2/22/97	Average	9/30-10/17/96	3/5-3/20/97	Average	Average
Persons/dwelling unit	2.36	2.47	2.42	2.74	2.73	2.74	2.58	2.34	2.46	2.81	2.78	2.80	2.78	2.67	2.73	2.34	2.49	2.42	
Baths	1.2	1.9	1.6	1.5	1.5	1.5	1.2	1.4	1.3	0.9	0.8	0.9	0.3	0.8	0.6	0.8	1.4	1.1	1.2
Showers	12.6	13.8	13.2	13.4	12.0	12.7	13.2	14.3	13.7	11.4	8.8	10.1	8.4	9.9	9.1	8.4	10.4	9.9	11.5
Clothes Washers	14.9	12.6	13.8	14.9	14.4	14.6	15.3	16.7	16.0	11.0	10.6	10.8	17.1	14.4	15.8	14.7	12.3	13.5	14.1
Dish Washers	1.7	1.2	1.4	1.0	1.2	1.1	1.3	1.2	1.3	0.9	0.7	0.8	1.0	0.7	0.8	0.7	0.5	0.6	1.0
Toilets	18.2	23.1	20.7	17.6	18.4	18.0	20.1	20.5	20.3	15.6	13.7	14.6	14.9	15.1	15.0	15.3	15.0	15.2	17.3
Faucets	10.7	10.9	10.8	9.4	9.0	9.2	10.2	9.9	10.1	8.2	6.3	7.2	10.1	10.4	10.3	10.4	10.9	10.7	9.7
Coolers	0.2	0.1	0.2	0.7	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0	0.1	0.1
Hot Tubs	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.1	0.0	0.0	0.0	0.0
Humidifiers	0.0	0.0	0.0	0.5	0.3	0.4	0.0	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Leaks	2.5	4.3	3.4	3.4	9.9	6.6	8.4	9.0	8.7	3.3	5.0	4.1	2.5	6.6	4.6	6.3	13.2	9.7	6.2
INDOOR	61.9	68.1	65.0	62.4	66.6	64.5	69.6	73.2	71.4	51.4	45.9	48.6	54.4	58.1	56.2	57.9	63.7	60.8	61.1
Irrigation	82.0	72.9	77.5	217.0	12.3	114.6	159.0	3.6	81.3	73.4	1.4	37.4	91.7	34.1	62.9	18.0	35.3	26.7	66.7
Swimming Pools	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.7	0.4	2.6	4.7	4.6	4.7	1.2
Unknown	1.4	0.3	0.8	1.1	0.8	0.9	1.0	1.6	1.3	0.9	0.7	0.8	1.2	1.0	1.1	2.5	2.4	2.4	1.2
OUTDOOR TOTAL	83.4	73.4	78.4	218.1	13.1	115.6	159.9	5.2	82.6	74.3	2.1	38.2	97.6	35.5	66.6	25.2	42.3	33.7	69.2
TOTAL	145.4	141.5	143.4	280.5	79.6	180.1	229.6	78.4	154.0	125.7	47.9	86.8	152.0	93.6	122.8	83.1	106.0	94.5	130.3

Table 3-3. Summary of indoor and outdoor water use in Phoenix, Scottsdale, Waterloo, Walnut Valley, Los Virgenes, and Lompoc

GALLONS PER CAPITA PER DAY

	PHOENIX, AZ			SCOTTSDALE/TEMPE, AZ			WATERLOO, ONT			WALNUT VALLEY, CA			LAS VIRGENES, CA			LOMPOC, CA			Six City
	4/29 - 5/16/97	11/4-11/19/97	Average	5/20-6/3/97	12/2-12/19/97	Average	6/23-7/10/97	10/7-10/22/97	Average	7/29-8/12/97	1/6-1/20/98	Average	8/19-9/3/97	1/27-2/10/98	Average	9/8-9/23/97	2/24-3/9/98	Average	Average
Persons/dwelling unit	2.77	2.92	2.85	2.25	2.42	2.34	3.09	3.15	3.12	3.33	3.23	3.28	3.1	3.04	3.07	2.79	2.83	2.81	2.9
Baths	0.8	1.1	1.0	1.1	0.8	1.0	1.8	0.9	1.4	1.4	0.7	1.1	1.4	1.2	1.3	1.1	1.7	1.4	1.2
Showers	12.6	11.2	11.9	12.0	11.5	11.8	7.5	8.8	8.2	11.0	12.2	11.6	9.5	12.6	11.1	12.1	10.8	11.5	11.0
Clothes Washers	14.0	15.6	14.8	13.2	14.2	13.7	12.2	13.2	12.7	13.2	14.6	13.9	15.3	17.3	16.3	16.2	15.3	15.8	14.5
Dish Washers	0.7	0.8	0.8	1.1	1.0	1.1	0.7	0.8	0.8	0.5	0.7	0.6	0.7	1.0	0.9	0.8	0.8	0.8	0.8
Toilets	17.1	16.9	17.0	16.9	17.2	17.1	16.4	18.6	17.5	15.6	16.6	16.1	13.3	15.1	14.2	15.3	13.9	14.6	16.1
Faucets	8.8	8.5	8.7	10.3	9.7	10.0	10.9	8.9	8.9	11.4	10.4	10.9	10.6	9.7	10.2	8.4	8.0	8.2	9.6
Coolers	2.5	0.5	1.5	4.7	0.2	2.5	0.0	0.0	0.0	0.0	0.0	0.0	0.1		0.1	0.0	0.0	0.0	0.7
Hot Tubs	0.0	0.1	0.1	0.1	0.3	0.2	0.0	0.0	0.0	0.5	0.1	0.3	0.5	0.0	0.3	0.1	0.0	0.1	0.1
Treatment	0.1	1.1	0.6	0.5	1.0	0.8	5.5	5.0	5.3	1.5	1.2	1.4	0.9	0.2	0.5	0.8	0.9	0.7	1.5
Leaks	14.6	10.3	12.5	13.8	15.8	14.8	7.3	5.1	6.2	6.9	7.8	7.4	9.3	9.2	9.3	8.0	8.9	8.5	9.8
INDOOR	71.2	66.1	68.7	73.7	71.7	72.7	62.3	61.3	61.8	62.0	64.3	63.2	61.6	66.3	63.9	62.6	60.4	61.5	65.3
Irrigation	138.8	68.2	103.5	234.6	70.1	152.4	24.1	1.0	12.6	154.4	11.8	83.1	288.4	33.7	160.0	60.7	13.0	36.8	91.4
Swimming Pools	4.5	2.7	3.6	10.4	6.6	8.5	0.0	0.0	0.0	3.5	0.2	1.9	10.5	2.4	6.4	0.0	0.0	0.0	3.4
Unknown	2.0	0.8	1.4	2.9	2.6	2.8	1.2	0.5	0.9	2.2	1.0	1.6	1.9	2.7	2.3	0.4	1.5	1.0	1.6
OUTDOOR TOTAL	145.3	71.7	108.5	247.9	79.3	163.6	25.3	1.5	13.4	160.1	13.0	86.6	298.8	38.7	168.8	61.1	14.5	37.8	98.4
TOTAL	216.5	137.8	177.2	321.6	151.0	236.3	87.6	62.8	75.2	222.1	77.3	149.7	360.3	105.1	232.7	123.7	74.9	99.3	161.7

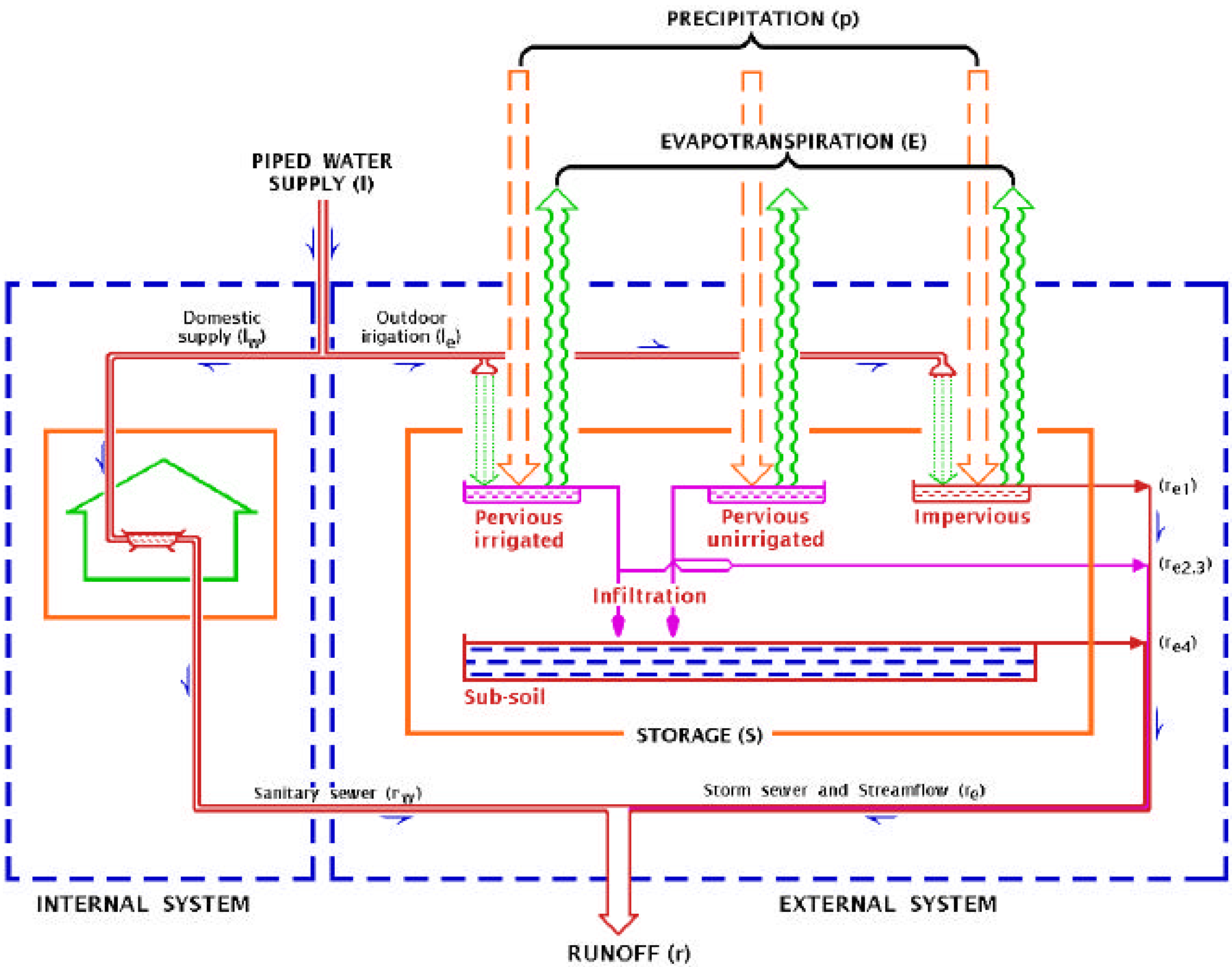


Figure 3-4. Hourly variability of indoor water use in 88 houses, Boulder, CO. (Harpring 1997).

Toilet Flushing: Toilet flushing is the most regular and predictable of all of the indoor water uses with an average of 16.7 gpcd and a range from 14.2 to 20.7 gpcd. Residents and guests will use the toilets every few hours if they are home. The only significant break in this pattern is during the night when people are asleep. Day to day variation in toilet flushing depends upon how many people are home at a given time. More people would be expected to be home on weekends and in the summer when school is not in session. Toilet flushing generates the black water that is the main source of pollutants at the wastewater treatment plant. The low variability of toilet use is good news from a design point of view since it is then only necessary to design for relatively small peaking factors. Also, low quality water can be used for toilet flushing. Thus, it is a good candidate for using reclaimed wastewater or stormwater.

Conservation options for toilets have focused on reducing the volume per flush from four to five gallons to 1.6 gallons which is mandated nationally in the plumbing codes. An important concern with regard to lower volume per flush is that people would double or triple flush. Based on a nationwide study of toilet flushing, Mayer et al. (1997) conclude that double flushing is a minor problem with low-flush toilets, occurring only about 6% of the time. Also, it does not appear that people will change their flushing patterns. British studies of the nature of toilet flushing indicate that only about 25 % of toilet flushes are to dispose of fecal material as shown in Table 3-4 (Friedler et al. 1996).

Table 3-4. Number of toilet flushes per day and proportion related to fecal flushes (Friedler et al. 1996)

Flushes/day	Week Day	Weekend Day
Fecal related	0.87	1.09
Other	2.24	2.43
Total	3.11	3.52

The diurnal pattern of fecal related flushes indicates that the majority take place between 6 am and 9 am. Thus, the savings result from fewer gallons per flush and not fewer flushes per day. The associated pollutant load would remain constant; accordingly, the wastewater concentrations would increase. Some concern exists that odors from sewers would be further intensified with the implementation of water conservation (Joyce 1995).

The volume per flush can be reduced to 0.5 gallons using pressurized systems. This technology may gain more widespread use in the future. Future toilets include the currently mandated low-flush (1.6 gallons) and ultra low-flush (0.5 gallons) conventional toilets. Johnson et al. (1997) describe an innovative toilet wherein feces and urine are collected in separate compartments. This toilet reduces water use and allows more efficient treatment of the two separate waste streams. Dual flush toilets are employed in Australia wherein the user selects whether to use more or less flushing water depending upon the need.

Clothes Washing: Clothes washers use an average of 14.3 gpcd with a range from 10.8 to 16.3 gpcd. The traditional Monday wash day has been replaced by a more uniform pattern of clothes washing which is done throughout the day with peaks in the morning and early afternoon as was shown in Figure 3-4. More efficient clothes washers are expected to reduce water use per load by about 25 percent. The timing on clothes washing could be affected by electric or water utility rates, which provide time of day incentives and disincentives. As mentioned earlier, water users in Great Britain apparently wash late at night to take advantage of lower electricity rates.

Showers and Baths: Showers (11.2 gpcd) are much more popular than baths (1.2 gpcd) for all 12 cities in the NAREUS study. For Boulder, CO, the morning shower is the predominant time for this activity as shown in Figure 3-5 (Harpring 1997). The other peak in showering occurs during the evening. Showers are taken on a daily basis in Boulder. Thus, no significant variability occurs from day to day. Drainage from showers can be used for lawn watering during the growing season of year. It is a significant source of reclaimable water and the timing of its entry into the wastewater collection system can be estimated accurately because the shower water is not stored during use.

The main conservation option for showers is to use low-flow shower heads. Results to date indicate only limited reduction in water use since users did not set the older shower heads to the higher flow rates. Federal law mandates a maximum flow rate for showers of 2.5 gallons per minute (gpm). Results of the NAREUS study indicate that most people set their shower flow rate below this level. Thus, conservation savings may not be that significant (Mayer et al. 1997). No significant change in duration of showers has been observed with the lower flow rate showers. Showers are also important as a major user of hot water.

Faucet Use: Faucet use includes drinking water, water for washing and rinsing dishes, flushing solids down the garbage disposal, shaving, and numerous other personal needs. Faucet use averages 9.3 gpcd with a range from 6.9 to 10.5 gpcd. No breakdown among these uses is available although one can make educated guesses as to the amounts of water used for these purposes. Best estimates of actual drinking water use are in the range of 1.0 to 2.0 liters per capita per day with a mean of 1.4 liters per day (Cantor et al. 1987). Garbage disposals add about one gpcd to total indoor consumption (Karpiscak et al. 1990). Faucet use requires the highest water quality because it is the potable water source. Overall, faucet use is a small proportion of total use, which suggests the possibility of separate treatment and distribution systems for this source. Also, faucet use is relatively common during the day so equalizing storage requirements are low.

Dishwashers: Dishwashers are a relatively minor water use and newer dishwashers are being designed to use less water to conserve energy and water.

Present per capita water use averages only 0.9 gpcd.

Water Use for Cooling: For some houses, and for many commercial and industrial establishments, water use for cooling is a significant part of the water budget. Swamp coolers are used in the more arid areas of the United States. Karpiscak et al. (1994) estimate that residential evaporative coolers use about six gpcd in Tucson, AZ. Because of the relatively small number of houses using coolers, the average usage is quite low, only 0.4 gpcd.

Outdoor Urban Residential Water Use

Whereas indoor residential water use is very constant across the United States and does not vary seasonally, irrigation water use varies widely from little use to being the dominant water use. Also, it varies seasonally. The 12 cities in the NAREUS are not a representative sample of the United States with regard to climate types. Also, the amount of natural precipitation that occurred during the study periods can have a significant impact on the results. Nevertheless, the results certainly suggest the potential major impact of irrigation on average and peak water use.

A detailed evaluation of irrigation water use as a potential reuse of urban stormwater is presented in Chapter 8. This section only introduces the subject. Irrigation water use follows a definite pattern of high use rates in the morning and evening with low use rates during the day and late at night. Thus, these customers are following the common recommendations to not water during the middle of the day. Watering late at night is discouraged because of the noise from the sprinklers.

For the entire NAREUS study, outdoor water use averaged 82.8 gpcd, significantly more than the indoor water use of 63.2 gpcd. Studies of overall residential water use in Boulder and Denver show that outdoor water use averaged over the entire year exceeds indoor water use. Thus, outdoor water use can be a significant component of total annual average water use.

For the NAREUS study, Waterloo, Ontario is representative of conditions in the northeastern part of North America. During the summer, the outdoor water use averaged 25.3 gpcd compared to indoor water use of 62.3 gpcd. As expected the outdoor water use became negligible in the colder months, averaging only 1.5 gpcd in October.

At the other extreme, outdoor water use in Las Virgenes, CA averaged 299 gpcd, nearly five times the indoor water use of 61.6 gpcd during the summer sampling period. Thus, for residential areas in the more arid and warmer parts of the country, lawn watering is the largest single use on an annual average basis and is the dominant component of peak daily and hourly use during the summer months.

In the arid areas, evapotranspiration requirements are much greater than natural rainfall. In warmer parts of the country, even those with abundant rainfall, such as Florida, irrigation water use rates are high because of the long growing season which includes some dry periods. Irrigation water use is a major input to the urban water budget during the growing season. A growing number of people are installing automatic sprinkling systems. These systems tend to use more water than manual systems (Mayer 1995). Also, the timers on these systems are seldom adjusted. Thus, lawn watering occurs even during rainy periods. Experience with soil moisture sensors to control sprinkling use has been mixed. Automatic sprinkling systems do offer the potential for more efficient use of water if they are properly calibrated and operated (Courtney 1997).

The hourly pattern of total residential water use (indoor plus outdoor) for Boulder, CO is shown in Figure 3-5 (Harpring 1997). The study period from late May to early June included some rainy days. Peak hourly use between 6 and 8 am is caused predominantly by irrigation. Comparison of Figures 3-4 (indoor only) and 4-5 (total) indicates the importance of irrigation. The indoor water use at 6 am is about 7.5 gallons per house while the total water use at the same time is about 41 gallons per house. Thus, irrigation constitutes over 80% of the peak hourly use.

Options for reducing outdoor water use include using less water-loving plants, applying water more efficiently, reducing the irrigated area, and using nonpotable water including stormwater runoff and treated wastewater (Courtney 1997). Irrigation use has an indirect effect on urban runoff because it causes much wetter antecedent conditions, which increases the portion of rainfall that runs off. Sakrison (1996) projects a potential decrease of 35% in the demand for irrigation water in King County, WA if the higher density urbanization occurs. For King County, the main way that water use is managed is by restrictions on outdoor water use for landscaping. A maximum permissible evapotranspiration is allotted that forces the property owner to reduce the amount of pervious area devoted to turf grass. Stormwater run-on to the pervious area can be used for an extra credit. The amounts of irrigable area for three typical single family lot sizes are shown below.

The advantage of clustering is obvious from inspection of Table 3-5. The amount of irrigable area per house is reduced from 5,000 sq. ft. to 1,500 sq. ft., a reduction of 70%. This is the main savings in water use. However, from a stormwater runoff point of view, the imperviousness would increase.

Table 3-5. Typical lot sizes and irrigable area, King County, WA (Sakrison 1996).

Density	Lot Size, (sq.ft.)	Irrigable Area Per lot (sq. ft.)	% of total
Low	10,000	5,000	50
Medium	7,000	3,000	43
High	4,500	1,500	33

Lawn watering has increased in the U.S. as population migration occurs to warmer, more arid areas. Also, urban sprawl means much larger irrigable area per dwelling unit. Lawn watering needs are a dominant component of peak water use in urban areas. Reuse of treated wastewater and stormwater for lawn watering appears to be a very attractive possibility for more sustainable communities.

Infiltration and Inflow

Infiltration and inflow are major issues in urban stormwater management. For example, the results of studies of Boulder, CO indicate that I/I is the major source of flow during high flow periods, which might cause SSOs (Heaney et al. 1996). Indeed, the actual sewage flow in the system is 8-10 mgd whereas flows reach 45-50 mgd during peak periods as shown in Figure 3-6. Thus, I/I is over four times the amount of legitimate dry weather flow (DWF). For Boulder, evidence exists that the I/I is clean ground water since pollutant concentrations drop as sewage flow increases. Thus, pollutant loads remain relatively constant. I/I is discussed in detail in Chapter 6.

Summary of Sources of Dry-Weather Flow into Sanitary and Combined Sewers

Based on a sampling of nearly 1,200 houses in 12 North American cities, in which flows were measured for four weeks in each house, very accurate information is available on indoor water use patterns. Indoor residential water use averages 63.2 gpcd and remains constant throughout the year. Commercial, industrial, and public uses need to be added to this amount to estimate total water use. Essentially all of the indoor water use enters the sanitary or combined sewers. Outdoor water use is an important, and highly variable, water use.

Outdoor water use exceeds indoor water use on an annual average in more arid parts of the country. It also the primary cause of peak summer water use, and can range as high as five to six times indoor water use during these periods. Because of its seasonal nature, outdoor water use is a major component of the peak design flow as is water for firefighting.

Water conservation practices can reduce water use significantly, particularly outdoor water use. The increasingly high cost of treating water should encourage a new look at dual water systems and more aggressive reuse systems. Infiltration and inflow are the main unknowns in designing sanitary sewer systems. I/I varies widely within a city and across cities. Contemporary practice still allows much higher peak flows to account for this uncertainty.

The primary source of degraded water quality for residential uses is toilet flushing which accounts for about 30% of the DWF. Faucet water is also of concern, especially where garbage grinders are used. Thus, about 50 % of the DWF could be classified as “blackwater”. The remaining sources including showers, baths, clotheswashers, and dishwashers would be classified as “graywater”. The largest source of illicit “wastewater” is I/I which can range from a small fraction to several times DWF.

The conclusion from this simple water budget is that only a small portion of the wastewater entering sewers requires a high level of treatment. The remaining water could receive less treatment or does not need treatment because it is probably the infiltration of clean groundwater. This mass balance indicates that innovative changes in current practices may be very cost-effective.

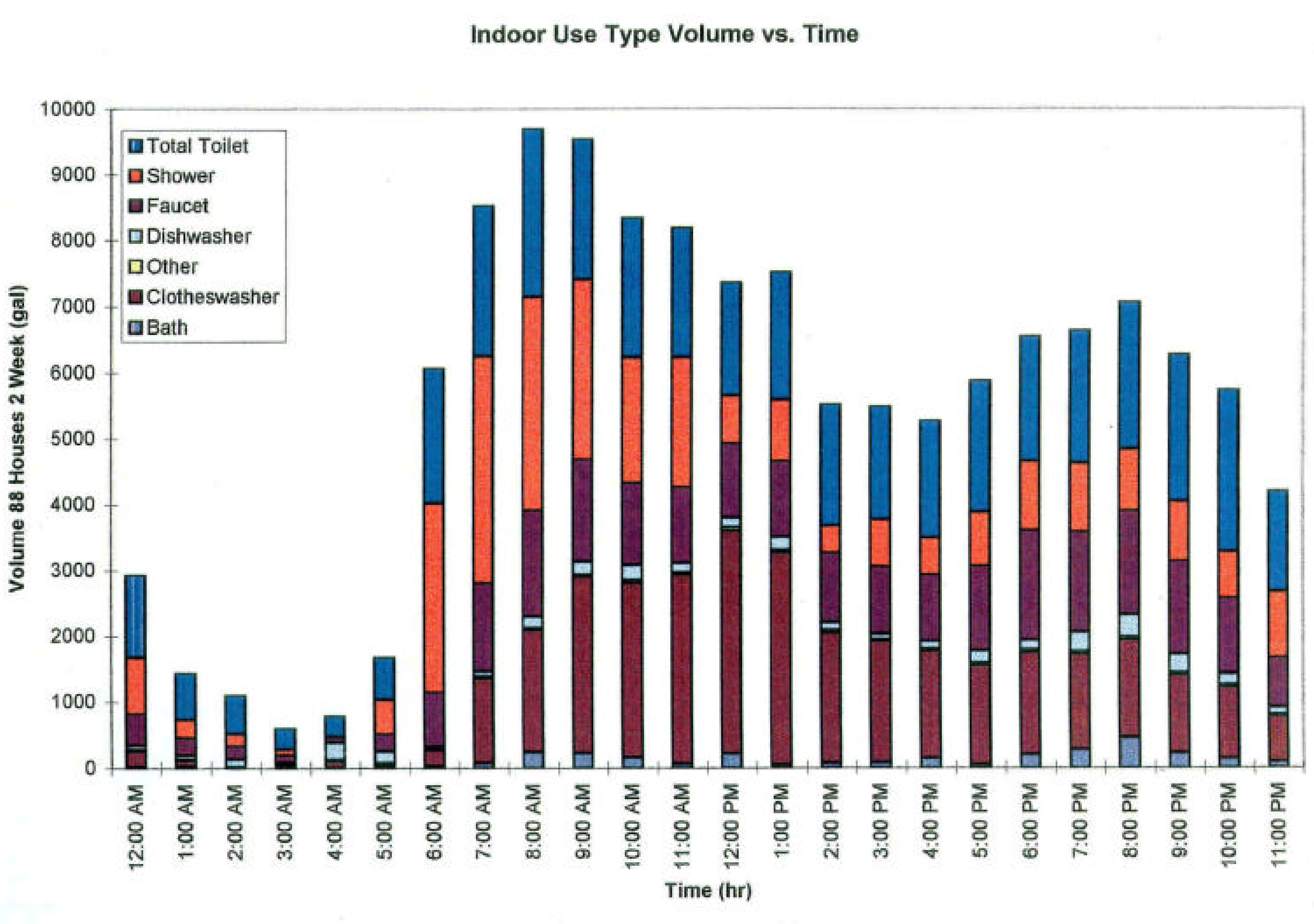


Figure 3-5. Weekday variability in total residential water use for 88 houses, Boulder, Co. (Harpring 1997).

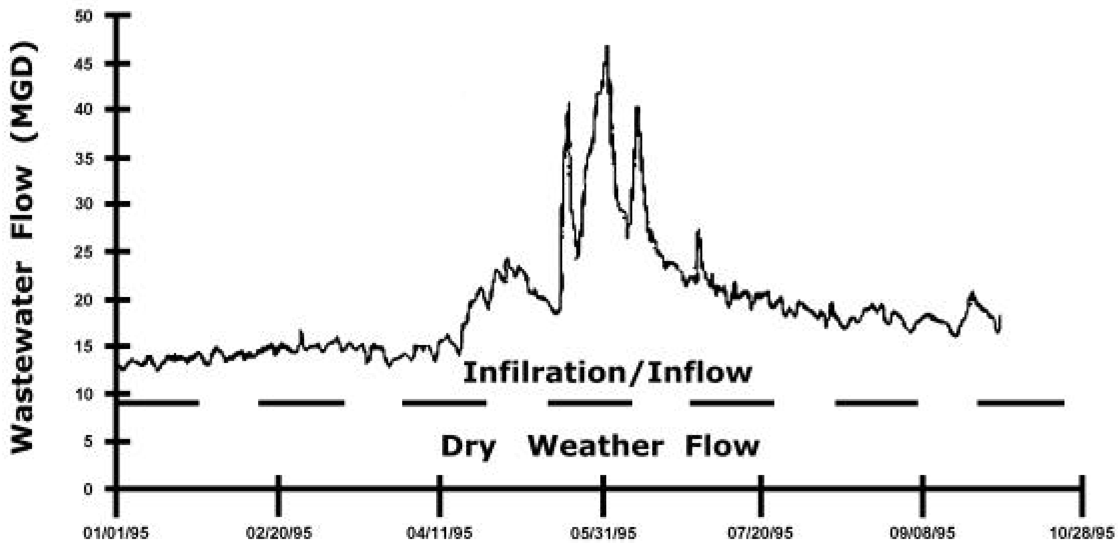


Figure 3-6. DWF., I/I and total wastewater flow, Boulder, CO, 1995 (Heaney et al. 1996).

Quantities of Precipitation in Urban Areas

Annual precipitation amounts for selected U.S. cities are listed in Table 3-6. The results of water budgets presented in the literature and water budgets for Denver and New York are presented in the remaining sections of this chapter.

Results of Water Budget Case Studies

Arizona

Two demonstration projects in Arizona provide examples of the results of aggressive water and energy conservation. The first project, which began in 1985, is located in Tucson, AZ, and is a demonstration house called Casa del Agua. The layout of the house and lot are shown in Figures 3-7 and 3-8 (Foster et al. 1988). Stormwater runoff from the impervious surfaces is directed to the adjacent pervious areas to provide supplemental irrigation water. Roof runoff is collected in rain cisterns with a total capacity of 14,000 gallons. Casa del Agua is a three bedroom, two-bathroom residence that has been retrofitted to incorporate low water use fixtures and water reuse systems. All graywater from the washing machine, tub, shower, lavatories, and one side of the kitchen sink is directed into a collection sump where it receives treatment (filtration) and then is stored until needed. Rainwater is a very high quality water source and is low in total dissolved solids making it ideal for use for evaporative cooling (Foster et al. 1988). It is also used for toilet flushing. The problem with rainwater, in Tucson, is that the supply is small and highly variable. The average annual precipitation for Tucson is only 11 inches (Karpiscak et al. 1990).

The baseline water use for an average Tucson house indicates a total daily use of 105 gallons per capita, of which 68 gpcd is for indoor use. All of this water is supplied from the municipal system. By comparison, the goal of the water conservation project was to import only 37 gpcd and to use 12 gpcd from rainwater and 30 gpcd from recycled graywater. The main reduction in indoor water use was to be achieved by flushing toilets with recycled water. The actual water use during the first year of the study was reduced by 33%. The total water use was broken down as follows: city water (77%), gray water (24%), and rain water (4%). Rainwater use was less than expected due to below average rainfall.

Graywater use was less than expected due to insufficient storage for gray water, necessitating its discharge periodically to the sanitary sewer. After four years of operation with some adaptation to improve performance, the use of municipal water was reduced by 66%. A key change was to convert one of the two 7,000 gallon rainwater collection tanks to a graywater storage tank (Karpiscak et al. 1990). As a result, very little graywater was discharged to the sanitary sewer system, greatly reducing the dry-weather wastewater flow to the WWTP. The use of graywater storage over the year indicates seasonal variability in the utilization with the storage full, or nearly full, in spring and then emptying during the main water use summer period of the year.

In addition to Casa del Agua, in Tucson, a newer demonstration house opened in Phoenix, AZ in May 1993. It is called Desert House and is located at the Desert Botanical Garden (Karpiscak et al. 1994). The goal of this demonstration house is to reduce energy and water use by 40%. This design will also focus on reducing peak summer water use. The main savings in indoor water use is due to reductions in toilet, shower, and washing machine use. The main reduction in outdoor water use results from using graywater for lawn watering. This 1,657 square foot, one story, single family
house is equipped with 1.5 gallon per flush toilets, 2.75 gallons per minute showerheads, and faucet aerators. Roof runoff goes to a 4,750 gallon cistern. The design size of the cistern had decreased significantly from the original size of 14,000 gallons in Casa del Agua. Desert House is designed for high visitor use so it is not operated in as routine a manner as Casa del Agua.

Table 3-6. Annual precipitation and days with rain for selected U.S. cities (US EPA 1979).

State	City	Region	Annual Precipitation, in.	Annual Days w/ Rain	Average in/day
AL	Birmingham	East	53.52	118	0.45
CT	Harford	East	42.43	128	0.33
FL	Miami	East	57.48	127	0.45
GA	Atlanta	East	47.14	115	0.41
KY	Louisville	East	41.47	122	0.34
LA	New Orleans	East	63.54	120	0.53
MA	Boston	East	42.77	128	0.33
MD	Baltimore	East	44.21	112	0.39
NC	Charlotte	East	43.38	110	0.39
NY	Buffalo	East	35.65	165	0.22
NY	New York	East	42.37	119	0.36
OH	Cincinnati	East	39.34	134	0.29
OH	Cleveland	East	32.08	156	0.21
PA	Pittsburg	East	36.87	146	0.25
PA	Philadelphia	East	42.48	115	0.37
TN	Nashville	East	45.00	120	0.38
IA	Des Moines	Midwest	31.06	105	0.30
IL	Chicago	Midwest	33.49	120	0.28
IN	Indianapolis	Midwest	39.69	124	0.32
MI	Detroit	Midwest	30.95	130	0.24
MN	Minneapolis	Midwest	24.78	113	0.22
MO	St. Louis	Midwest	36.46	104	0.35
MO	Kansas City	Midwest	34.07	98	0.35
NE	Omaha	Midwest	25.90	94	0.28
TX	Austin	Midwest	32.58	81	0.40
TX	Dallas	Midwest	34.55	80	0.43
TX	Houston	Midwest	45.26	103	0.44
WI	Milwaukee	Midwest	27.57	119	0.23
CO	Boulder	Rocky Mtn.	18.57	87	0.21
NM	Albuquerque	Rocky Mtn.	8.13	58	0.14
UT	Salt Lake City	Rocky Mtn.	14.74	87	0.17
A K	Anchorage	West	14.71	126	0.12
AZ	Phoenix	West	7.42	34	0.22
CA	Los Angeles	West	14.62	35	0.42
CA	San Francisco	West	20.78	67	0.31
DC	Washington	West	40.78	107	0.38
HI	Honolulu	West	23.96	99	0.24
NV	Las Vegas	West	4.35	25	0.17
OR	Portland	West	39.91	149	0.27
WA	Seattle	West	34.10	164	0.21
WA	Spokane	West	17.19	118	0.15
		Mean	33.30	109	0.31
		Max	63.54	165	0.53
		min	4.35	25	0.12

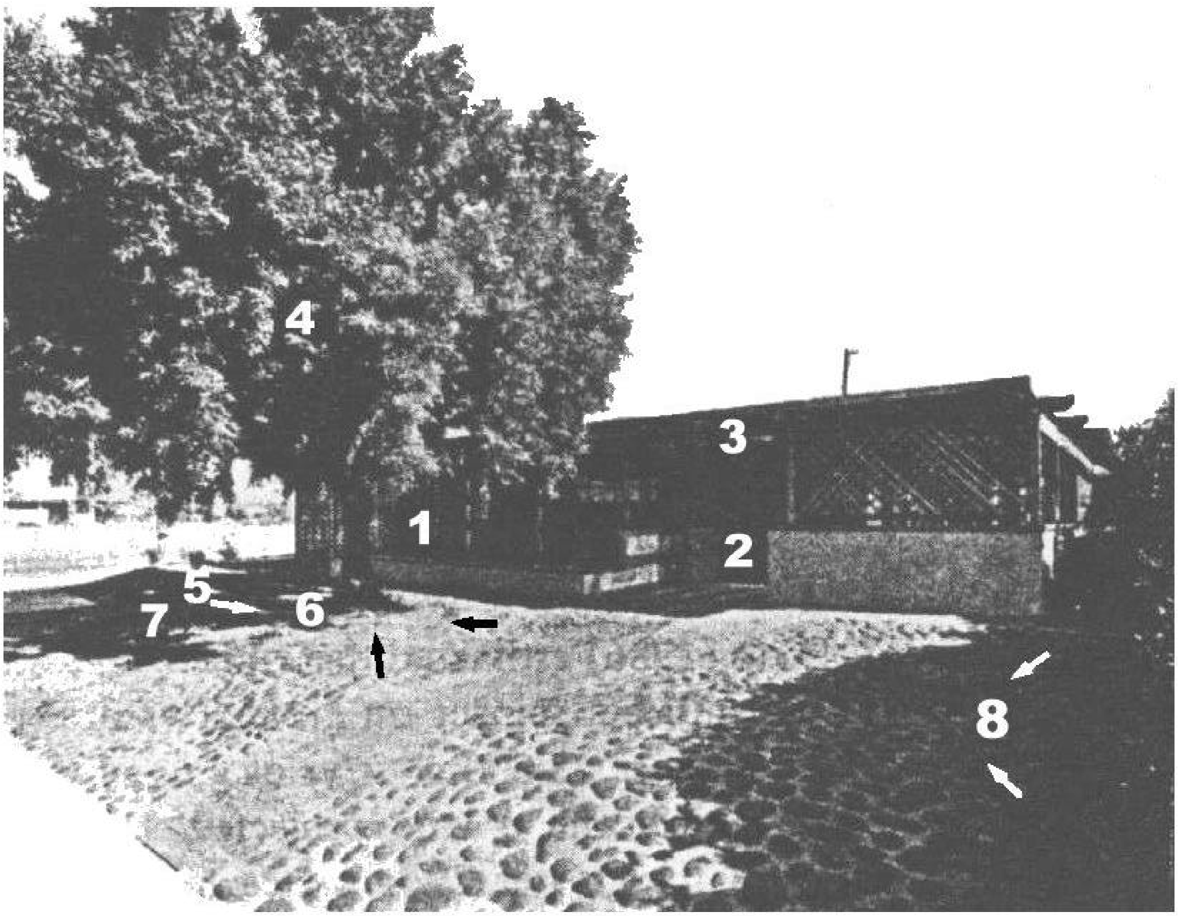


Figure 3-7. Front yard of Casa del Agua (Foster et al. 1988)
1) grape vine will grow over the lattice for more complete shade, 2) main entry defined and visually separated from street/driveway, 3) reed covered entry arbor provides shade from the west sun, 4) *Rhus lancea,* 5) *Cassia phyllodinea,* 6) *Lantana montevidensis,* 7) perimeter of yard is bermed to contain the rain and direct it to the plants, 8) cobblestone driveway directs rain to plants.

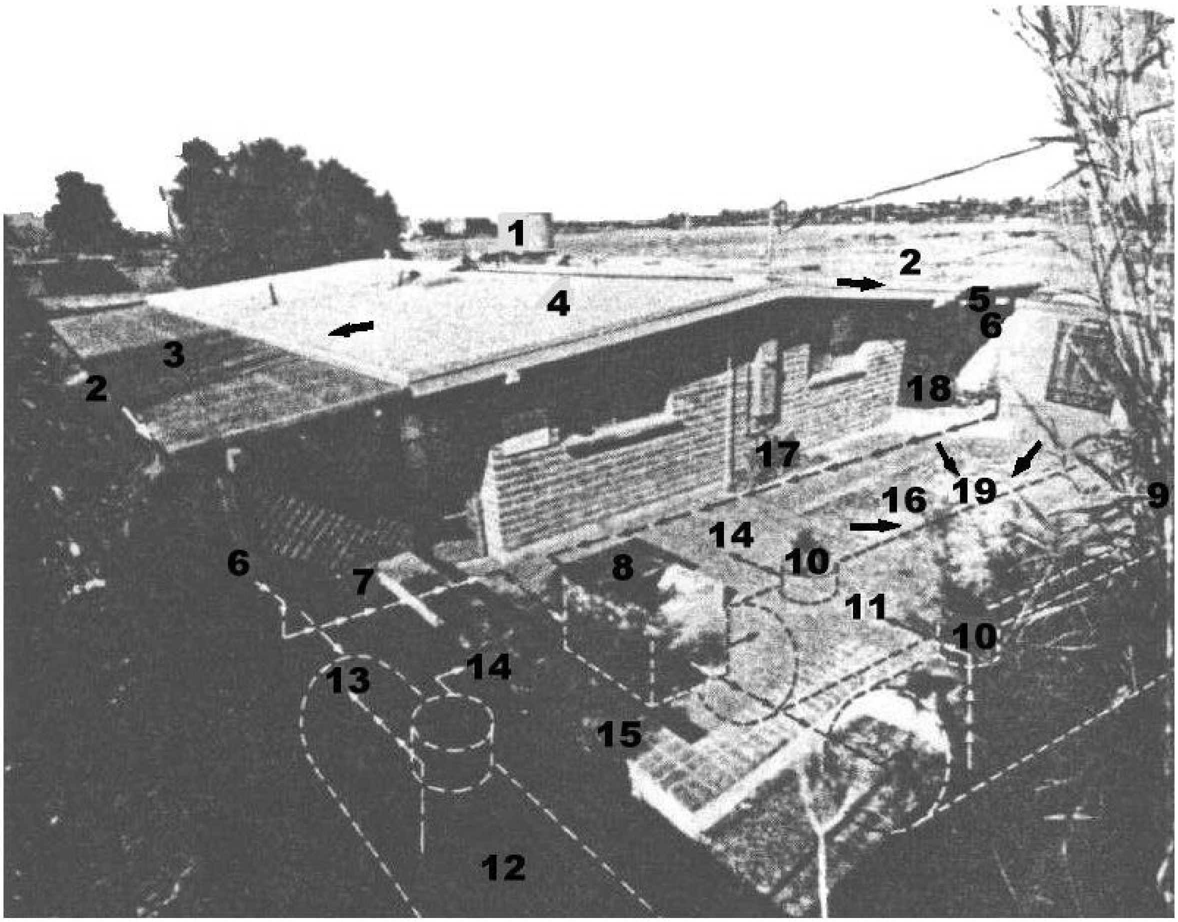


Figure 3-8. Back yard of Casa del Agua (Foster et al. 1988).
1) evaporative cooler, 2) new aluminum gutter, 3) new filon greenhouse roof, 4) existing gravel roof, 5) new filon porch roof, 6) new aluminum downspout, 7) pipe from downspout to filter, 8) concrete filter box with screen, 9) rain cisterns (14,000 gallons total), 10) cistern access, 11) supply to pump, 12) graywater cistern (800 gallons), 13) supply to pump, 14) overflow to sewer, 15) seat walls, 16) brick paving defined and visually separated from street/driveway, 17) *kalanchoe species,* 18) herb garden, 19) *Acacia pennacula*.

Germany

According to Grottker and Ottterpohl (1996), "the separation of feces and urine from the domestic waste water is identified as the most important step to a sustainable water concept." A 100-unit housing complex in Lubeck-Flintenbreite, Germany is being built using this concept. Key components of this innovative project are:

1. Storm water of private properties is re-used for toilet flushing, washing-

machines and irrigation in gardens. The overflow of stormwater storage is connected to the infiltration trenches of the road drainage. Two advantages of this approach are less potable water consumption and less detergent consumption.

2. Storm water from roads and other public surfaces is drained by infiltration depressions with trenches to the small creek. This method increases evaporation and retention of storm runoff.

3. Graywater is treated in aerated sand filters or constructed wetlands. The overflow is connected to the infiltration trenches of the public road drainage. Two advantages of this approach are using a simple treatment technique with high efficiency and waste water runoff retention.

4. Feces and other organic matter from households are transported by a vacuum system to a semi-central aerobic reactor with sludge storage, where the organic matter of 100 living units is treated. Vacuum toilets are used as inlets. Further, collected organic matter/waste is added to the anaerobic reactor. The treated sludge is stored and later carried to a farm. Three advantages of this approach are no I/I problem, less pollution in the treated sludge yields very high fertilizer, and biogas can be used in a semi-central heating system

This new system will be completely monitored for two years to do a final evaluation.

Melborne, Australia

Mitchell et al. (1996) used a daily water budget simulation model to evaluate the impact of on-site water management. They evaluated water use for two blocks in Melbourne, Australia. The attributes of each block are shown in Table 3-7.

Table 3-7. Attributes of two neighborhoods in Melbourne, Australia (Mitchell et al. 1996).

Attribute	**Neighborhood**	
	Essendon	**Scoresby**
Rainfall, mm/yr	591	887
Rain, days/yr	196	215
Evaporative demand, mm/yr.	1054	1054
Soil Type	clay	silty clay
Area, sq m	750	750
Roof plan area, sq m	203	203
Paved area, sq m	113	113
Garden area, sq m	434	434
People/house	3	3
Type of garden	standard	standard

The following retrofits were evaluated in these two areas:

- 13 kiloliter rain tank for storage of roof runoff for laundry, toilet, and garden water uses. Spillage is directed to the storm drainage network.
- Graywater from bathrooms and laundry is used for gardening through a sub-surface irrigation system. Overflows go to the wastewater sewer.

The simulated performance of the modified system is summarized in Table 3-8 (Mitchell et al. 1996).

Table 3-8 Simulated performance of modified urban systems (Mitchell et al. 1996).

Attribute	Neighborhood	
	Essendon	Scoresby
Water demand, kl/yr	278	265
Reduced demand for imported water, %	41	49
Reduced off-site stormwater runoff, %	56	49
Reduced wastewater runoff, %	11	8
Usage from rainwater tank, kl/yr	84	107
Rain tank deficit/demand	0.48	0.3
Use of graywater, kl/yr	28	24
Graywater deficit/demand	0.65	0.65

The reduction in demand for imported water was 41 and 49% for the two systems while off-site stormwater runoff was reduced by 56 and 49% for the two neighborhoods. These results indicate the potentially major impact of on-site water management on overall water use.

Adelaide, Australia

Adelaide is typical of other cities in that the water supply, wastewater, and stormwater infrastructure systems have developed independently of each other and now exist as large centralized systems. Adelaide has a separate sewer system. The demand for water in Adelaide, shown in Figure 3-9, indicates that direct contact needs are about 52 GL/a, 8 GL/a for process and manufacturing, 82 GL/a for gardens and other irrigation, and 18 GL/a for toilet flushing, or a total of 157 Gl/a. Thus, the majority of the water demand does not require high quality water. The potentially available local supply, shown in Figure 3-10, indicates 30GL/a from roof runoff, 95 GL/a from hillside runoff, 61 GL/a from street runoff, 52 GL/a from graywater effluent, and 24 GL/a from blackwater effluent, or a total of 260 GL/a. Thus, on the average, the potential local supply exceeds the demand, and the possibility exists for a locally sustainable system if the necessary storage, treatment, and redistribution facilities could be provided.

The monthly variability in demand, rural runoff, effluent, and urban runoff are shown in Figure 3-11. The present centralized system utilizes 550 kl per person in storage. According to calculations of Clark et al. (1997), the decentralized system would require only 150 kl per person to provide adequate water during a one in a 100 year drought. The overall proposed water budget components for the Adelaide system is shown in Figure 3-12.

Urban wastewater is being reused at several locations in Australia, (e.g., Rouse Hill near Sydney), with a first stage of 25,000 dwellings (Law 1997) and on a small scale at New Haven Village in Adelaide with 67 dwellings. New Haven Village is an innovative development of 65 medium density affordable dwellings that is designed as an implementation of the integrated approach (Clark et al. 1997). Key water management features include on-site treatment and reuse of household effluent, an innovative stormwater drainage system, and demonstration technology for an underground sub-surface irrigation system. With on-site treatment and reuse of household sewage and stormwater runoff, virtually no water leaves the site. The wastewater plant is located underground. Treated water is used for irrigation and toilet flushing, thereby reducing water demand by 50%. Two 22,500 liter underground storage tanks provide effluent storage. Sludge is disposed to a sludge thickening plant on site. Street widths have been reduced from 12.4 meters to only 6.8 meters. The stormwater is captured in a 40,000 liter underground concrete tank. Overflows go to an infiltration trench, and finally to a retention area for extremely heavy rainfalls. The tank delivers stormwater to the treatment plant at night for treatment.

Other larger demonstration projects are underway in Australia. Notable projects include New Brompton Estate in which roof runoff is being stored in an underground aquifer. Overall, the studies by Clark et al. (1997) demonstrate the feasibility of water self-sufficiency for the City of Adelaide with an annual rainfall of 600 mm, which is typical of average rainfall conditions in the United States.

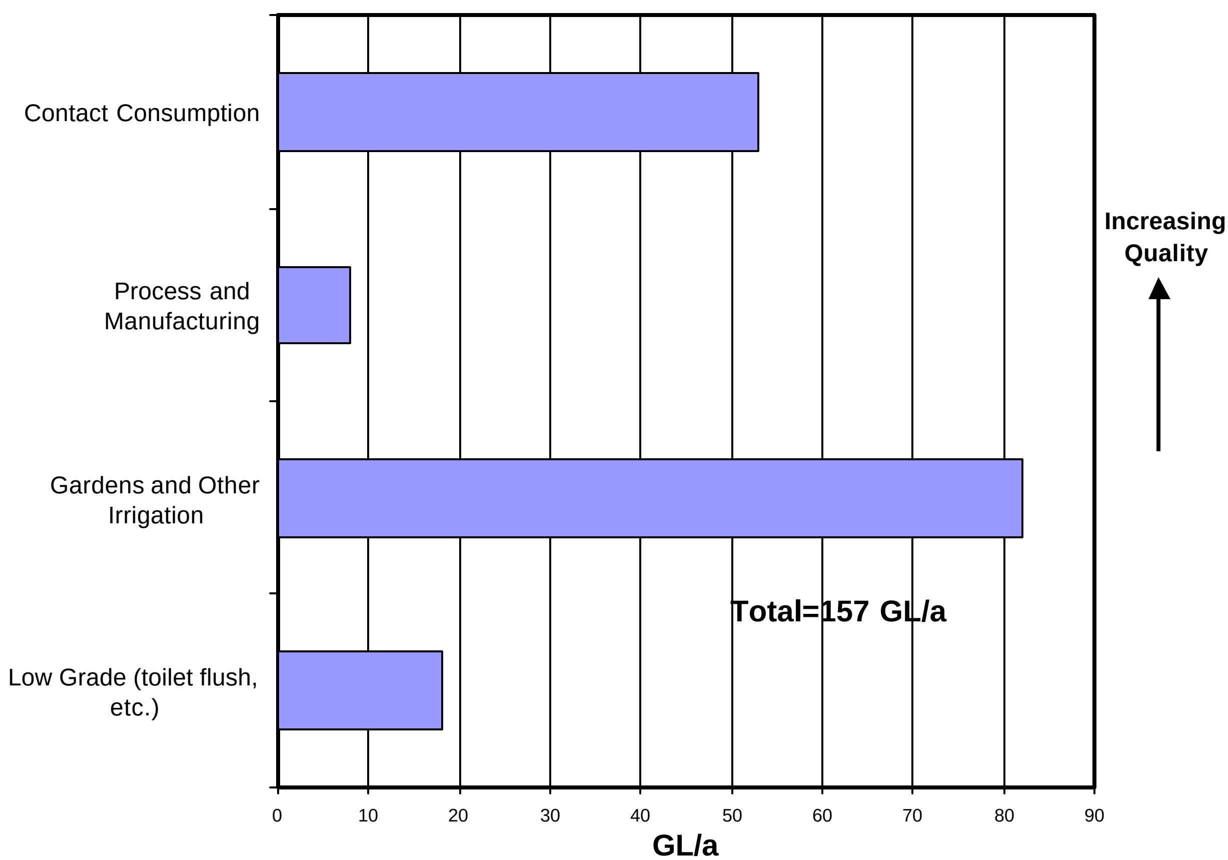


Figure 3-9. Consumption of water in Adelaide, Australia according to quality (Clark, 1997).

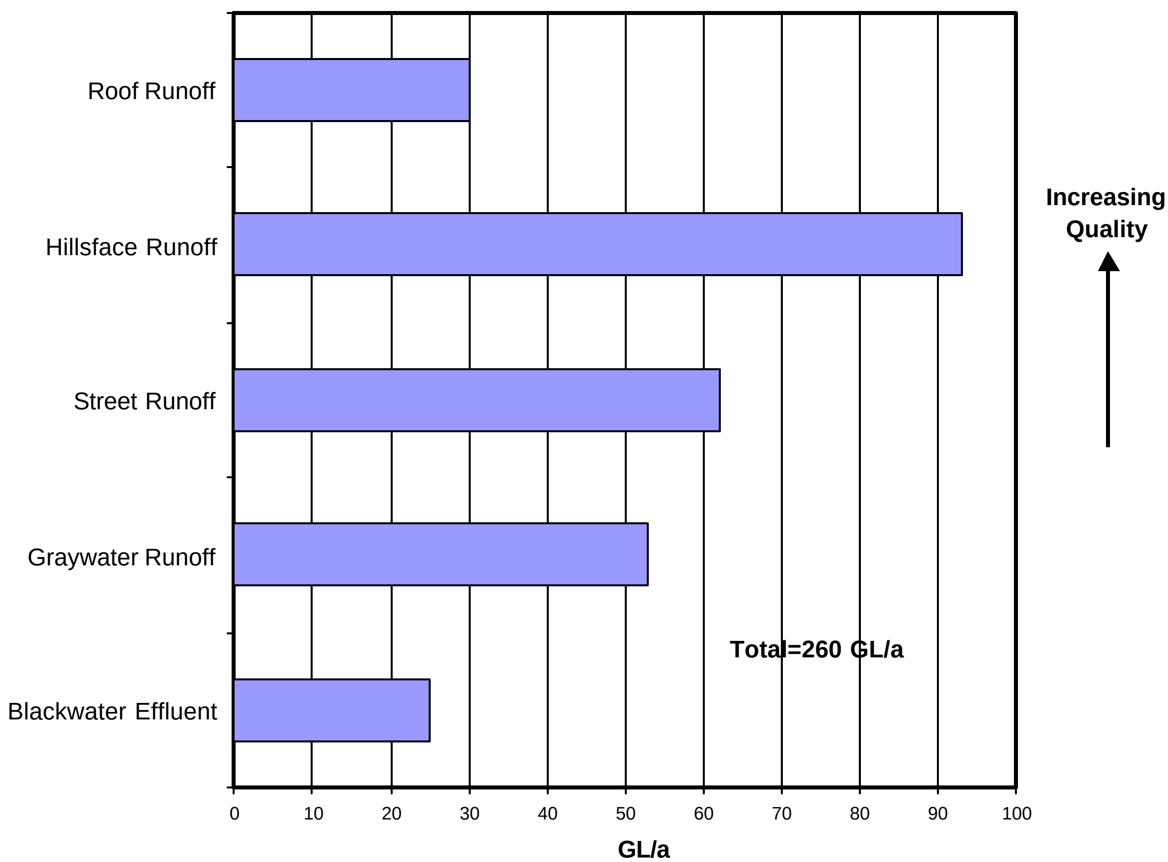


Figure 3-10. Availability of wastewaters in Adelaide, Australia according to quality (Clark, 1997).

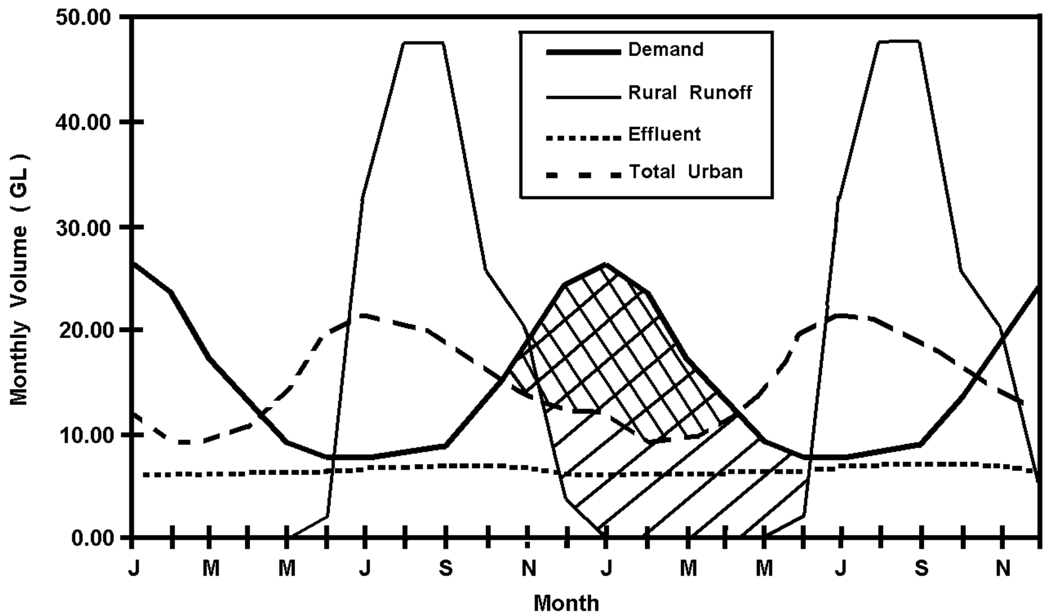


Figure 3-11. Typical monthly water supply and demand, Adelaide, Australia (Clark 1997).

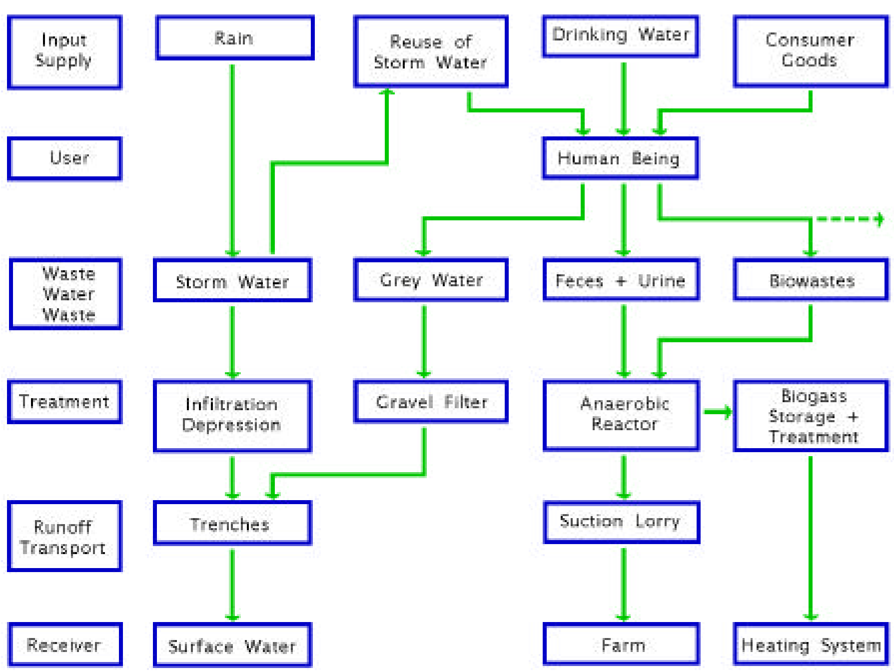


Figure 3-12. Flow chart of proposed integrated water system for Adelaide, Australia (Clark et al. 1997).

Simulated Monthly Urban Water Budgets for Denver and New York

General

This section presents the results of monthly simulations of water budgets for cities with climates similar to New York and Denver. The results should not be construed to be accurate representations of actual conditions in these two cities. The purpose of presenting these case studies is to show the relative importance of the various terms in the water budget and to show the impact of climatic conditions. The common assumptions for the comparative studies of representative urban neighborhoods in Denver and New York are presented in Table 3-9.

Table 3-9. Assumed common attributes of representative neighborhoods in Denver, CO and New York, NY.

Area, acres	Impervious Area Total	Impervious Area Directly Connected	100
Roof area, acres	15	5	
Driveway area, acres	10	5	
Local street area, acres	10	10	
Major street area, acres	5	5	
Lawn area, acres			60
Directly connected imperviousness, DCI, %			25
People			1,000

Water Use

Indoor Water Use

Assumed per capita water use estimates for the two cities are shown in Table 3-10.

Table 3-10. Assumed indoor water use for Denver, CO and New York, NY neighborhoods.

Item	Flow (gpcd)	% of Total	Black Water (gpcd)	Gray Water (gpcd)
Toilets	16	26.6	16	
Showers	10	16.7	10	
Baths	1	1.7	1	
Faucet-drinking	1	1.7	1	
Faucet-other	9	15.0	9	
Dishwashers	2	3.3	2	
Clothes washers	14	23.3	14	
Leaks	7	11.7	1	6
Total	60	100	54	6

The land use for the two representative neighborhoods is typical low density residential. The same population density of 10 persons per acre is used for the Denver and New York because since the purpose of this exercise is to illustrate the impact of rainfall and climate.

Outdoor Water Use

The estimated outdoor water use for the two cities is shown in Table 3-11.

Table 3-11. Estimated monthly outdoor water use in Denver, CO and New York, NY.

Month	Denver (gpcd)	New York (gpcd)
1	0	0
2	0	0
3	15	0
4	50	0
5	90	40
6	175	70
7	210	100
8	175	70
9	70	30
10	20	0
11	0	0
12	0	0
Mean	67	26

Inspection of Table 3-11 indicates that the per capita outdoor water use of 67 gpcd for this prototype area in Denver exceeds the indoor water use of 60 gpcd whereas average annual outdoor water use on New York of 26 gpcd is less than one half of the indoor water use because New York receives more annual precipitation and has lower evapotranspiration needs than Denver. Peak water use occurs during the summer in both locations and most of that peak is caused by lawn watering. Denver's peak monthly outdoor water use of 210 gpcd is over three times the indoor water use during July. Thus, urban lawn watering is the dominant component in peak water use in most urban areas. Peak water use is an important factor in sizing water infrastructure.

Total Water Use

Total water use (indoor plus outdoor) for Denver and New York is shown in Table 3-12.

Table 3-12. Total monthly water use for representative residential areas in Denver, CO and New York, NY.

Total Water Use for Denver

Month	Black Water (gpcd)	Gray Water (gpcd)	Total (gpcd)	Outdoor (gpcd)	Total (gpcd)
1	17	43	60	0	60
2	17	43	60	0	60
3	17	43	60	15	75
4	17	43	60	50	110
5	17	43	60	90	150
6	17	43	60	175	235
7	17	43	60	210	270
8	17	43	60	175	235
9	17	43	60	70	130
10	17	43	60	20	80
11	17	43	60	0	60
12	17	43	60	0	60
Mean	17	43	60	67	127

Total Water Use for New York

Month	Black Water (gpcd)	Gray Water (gpcd)	Total (gpcd)	Outdoor (gpcd)	Total (gpcd)
1	17	43	60	0	60
2	17	43	60	0	60
3	17	43	60	0	60
4	17	43	60	0	60
5	17	43	60	40	100
6	17	43	60	70	130
7	17	43	60	100	160
8	17	43	60	70	130
9	17	43	60	30	90
10	17	43	60	0	60
11	17	43	60	0	60
12	17	43	60	0	60
Mean		43	60	26	86

Histograms of monthly water use for Denver and New York are shown in Figures 3-13 and 3-14. Per capita indoor residential water use is the same for the two cities with only 17 gpcd of the water use producing black water and 43 gpcd of gray water. There is very little monthly variability in indoor water use. On the other hand, outdoor water use varies widely over the year and is the predominant cause of peak water use.

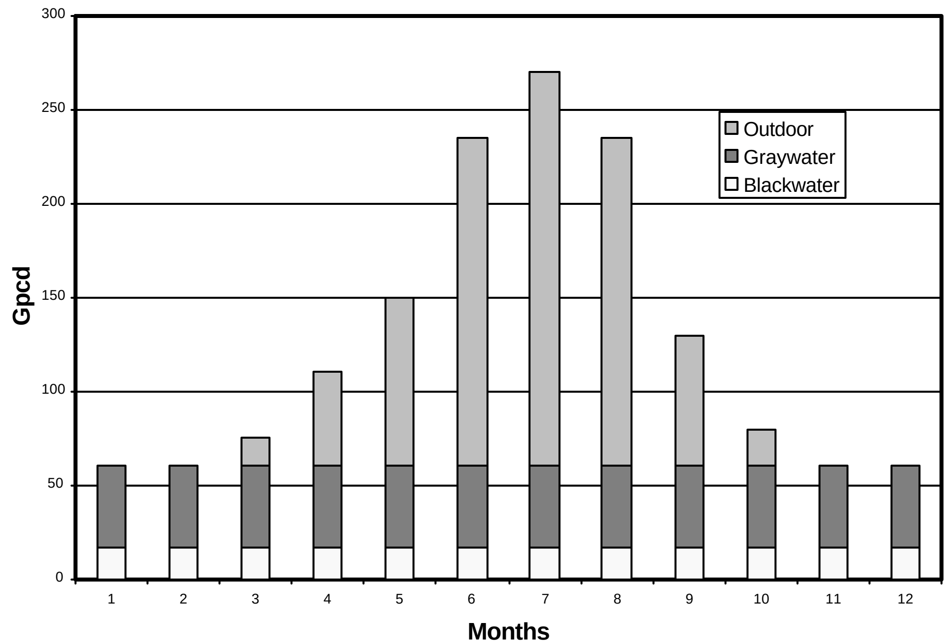


Figure 3-13. Average water use, Denver, CO.

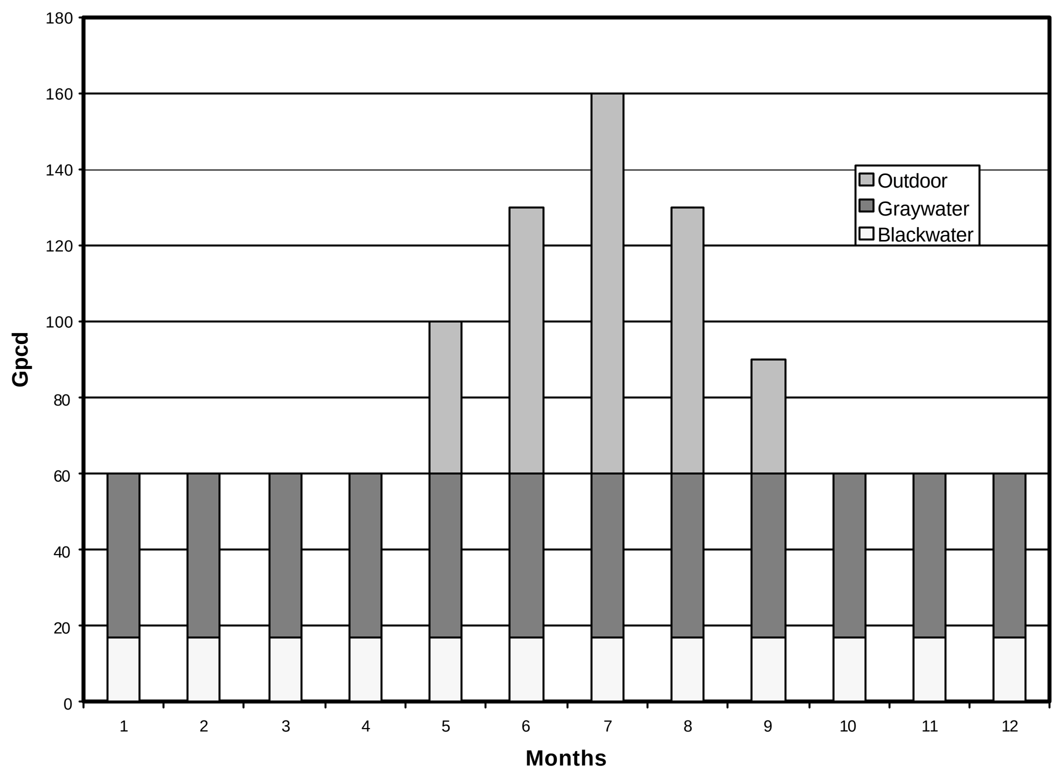


Figure 3-14. Average water use, New York, NY.

Wastewater

Wastewater or DWF = indoor water use residual + I/I. Nearly all of indoor water use enters the sanitary sewer system. Small losses in indoor water use, (e.g., from taking water on a picnic), are probably offset by the discharge to the sewer system of fluids brought into the house that are poured down the drains (e.g. leftover soft drink). Thus, it is reasonable to assume that 100% of the indoor water use, or its equivalent, enters the wastewater system. Salient assumptions used in the Denver-New York analysis are:

- Indoor water use residuals: Assume 100% of indoor water use goes to the sanitary or combined sewer. This component is DWF.
- Infiltration/inflow: I/I = base infiltration + rain-induced inflow and infiltration. Infiltration varies widely depending on construction and maintenance practices. Sanitary sewers are designed for two to six times DWF with the base sewer infiltration assumed to be 60 gpcd. Rain-induced infiltration, in gpcd, is computed as follows:
    - Denver, I = 60 times P(monthly inches)
    - New York, I = 20 times P(monthly inches)

The estimated I/I is presented for illustrative purposes and does not necessarily represent actual I/I for these two cities.

The total estimated wastewater flows for Denver, CO and New York, NY are shown in Figures 3-15 and 3-16 and Table 3-13. As with indoor and outdoor water use, black water and gray water associated with indoor water use are essentially constant throughout the year. However, I/I varies widely over the year and determines the design capacity for the wastewater network. Traditionally, I/I has been accepted as part of normal sewer flows. This topic is evaluated in Chapter 6.

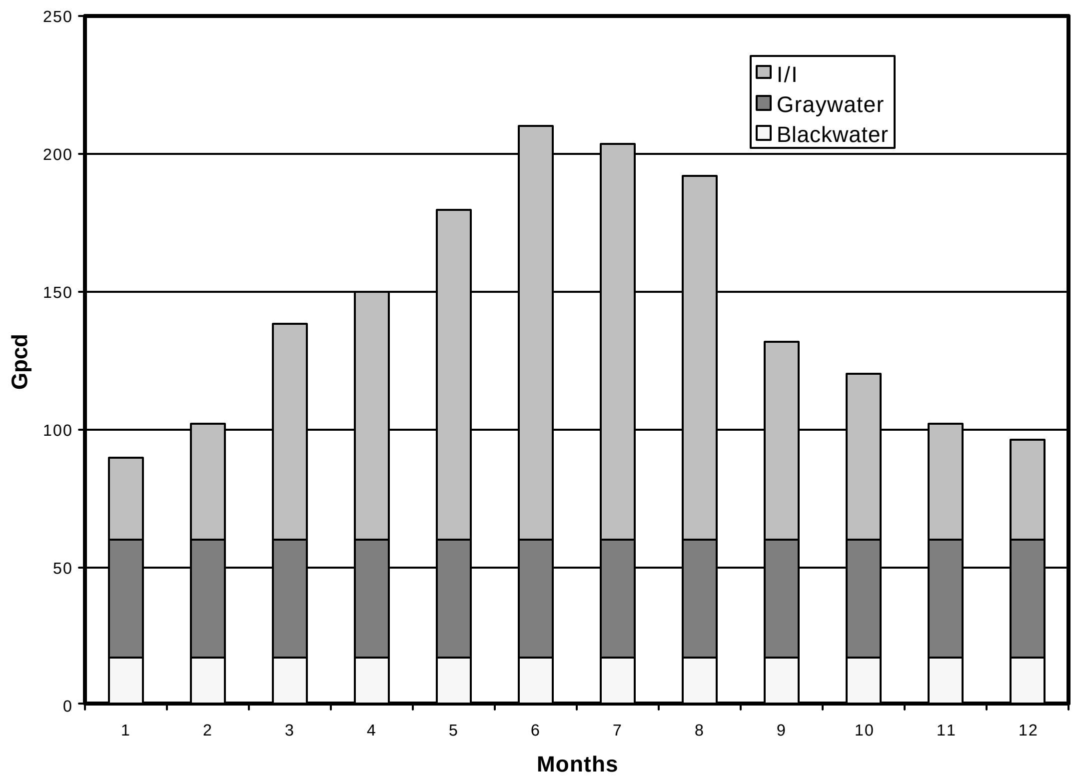


Figure 3-15. Monthly residential wastewater discharge, Denver, CO.

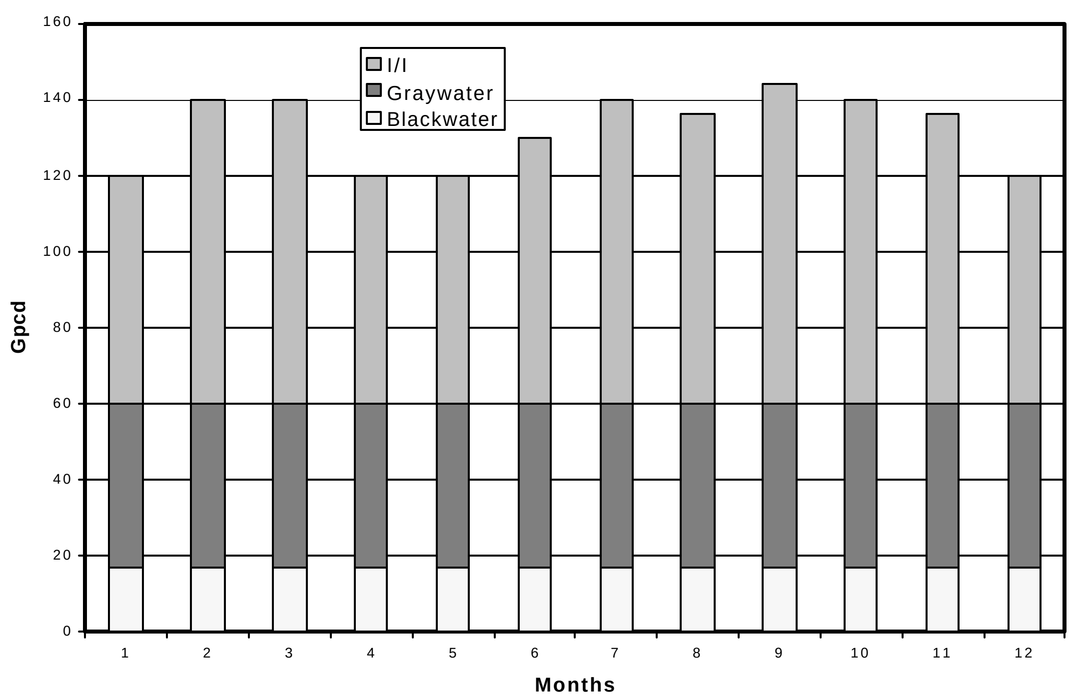


Figure 3-16. Monthly residential wastewater discharge, New York, NY.

Table 3-13. Total monthly wastewater flows for Denver, CO and New York, NY.

Denver

Month	Precip. (inches)	Black Water (gpcd)	Gray Water (gpcd)	I/I (gpcd)	Total (gpcd)
1	0.5	17	43	30	90
2	0.7	17	43	42	102
3	1.3	17	43	78	138
4	1.5	17	43	90	150
5	2.0	17	43	120	180
6	2.5	17	43	150	210
7	2.4	17	43	144	204
8	2.2	17	43	132	192
9	1.2	17	43	72	132
10	1.0	17	43	60	120
11	0.7	17	43	42	102
12	0.6	17	43	36	96
Total	17.0				
Mean	1.38	17	43	83	143

New York

Month	Precip. (inches)	Black Water (gpcd)	Gray Water (gpcd)	I/I (gpcd)	Total (gpcd)
1	3.0	17	43	60	120
2	4.0	17	43	80	97
3	4.0	17	43	80	97
4	3.0	17	43	60	77
5	3.0	17	43	60	77
6	3.5	17	43	70	87
7	4.0	17	43	80	97
8	3.8	17	43	76	93
9	4.2	17	43	84	101
10	4.0	17	43	80	97
11	3.8	17	43	76	93
12	3.0	17	43	60	77
Total	43.0				
Mean	3.61	17	43	72	93

Stormwater Runoff

The final component of the urban water budget to be estimated is the quantity of stormwater runoff. General characteristics of the study areas were shown in Table 3-9.

The runoff volume, R, from precipitation, P, is estimated as $R = C*P$ where C = runoff coefficient. This coefficient is assumed to equal the directly connected imperviousness, I. For this example, I = 0.25. The estimated monthly precipitation and runoff for Denver and New York are shown in Table 3-14.

Table 3-14. Monthly precipitation and runoff for Denver, CO and New York, NY.

Month	Denver		New York	
	Precipitation (inches)	Runoff (inches)	Precipitation (inches)	Runoff (inches)
1	0.5	0.13	3.0	0.75
2	0.7	0.18	4.0	1.00
3	1.3	0.33	4.0	1.00
4	1.5	0.38	3.0	0.75
5	2.0	0.50	3.0	0.75
6	2.5	0.63	3.5	0.88
7	2.4	0.60	4.0	1.00
8	2.2	0.55	3.8	0.95
9	1.2	0.30	4.2	1.05
10	1.0	0.25	4.0	1.00
11	0.7	0.18	3.8	0.95
12	0.6	0.15	3.0	0.75
Total	16.6	4.15	43.3	10.83

Summary Water Budgets

Water use and wastewater flows are typically expressed in terms of gallons per day. Stormwater runoff is usually expressed in inches averaged over the entire catchment. All flows were converted to inches averaged over the 100 acre catchment with 1,000 residents. The common assumed values, presented earlier in this analysis, are:

1. Population — 1,000
2. Area, acres — 100
3. Indoor water use, gpcd — 60
4. Runoff coefficient — 0.25
5. Conversion factors: — 7.48 gallons = 1 cu ft; 43,560 sq ft = 1 acre

The summary results for Denver, CO and New York, NY are presented in Tables 3-15 and 3-16. Denver results indicate a natural input from precipitation of 16.6 inches per year and imported water of 17.15 inches per year, slightly more than the natural input. The majority of the imported water is used for lawn watering. On the output side for Denver, I/I at 11.19 inches is the largest source of the 19.26 inches of water going to the WWTP. Urban runoff contributes an additional 4.15 inches of water leaving the system. Nearly 40% of the urban runoff falls on roofs and driveways. A good portion of that water could be retained on-site and infiltrated and/or used for lawn watering. Urban runoff alone is insufficient to provide sufficient water for lawn watering. However, urban runoff and graywater do provide enough water to meet essentially all of the lawn watering needs.

New York results indicate a natural input from precipitation of 43.3 inches per year and imported water of 11.57 inches per year, slightly more than a quarter of the natural input. The majority of the imported water is used for indoor purposes. On the output side for New York, I/I at 9.69 inches is the largest source of the 17.76 inches of water going to the WWTP. Urban runoff contributes an additional 10.83 inches of water leaving the system. Nearly 40% of the urban runoff falls on roofs and driveways. A good portion of that water could be retained on-site and infiltrated and/or used for lawn watering. Urban runoff alone is sufficient to provide sufficient water for lawn watering.

Future Urban Water Scenarios

Future scenarios for urban water use and wastewater discharges include combinations of the following futures. Water use estimates in gallons per capita per day include the pro rata additional nonresidential use, which is included in the per capita figure.

- Status Quo: This scenario means continuing the current pattern of water use and wastewater disposal. The nationally mandated compulsory use of low flush toilets should reduce per capita consumption by 10-15 gpcd. Legitimate sewage quantities should be in the 75-90 gpcd range. This per capita figure includes the added water use of non-residential customers averaged over the residential population. I/I would add another 50 to 400 gpcd to these flows. Solids loading will remain the same; thus, DWF concentrations will increase accordingly.
- Significant indoor water conservation: This scenario means replacing existing plumbing systems with water conserving devices including low-flush toilets, low flow rate shower heads, lower water using appliances. Expected sewage quantities are in the 50-65 gpcd range. Some I/I control is expected which reduces I/I to 25 to 300 gpcd. Increased DWF concentrations are expected.
- Gray water systems with aggressive I/I control: This scenario is defined as the preceding scenario with on-site use of gray water for lawn watering and toilet flushing. Expected sewage quantities are in the 30-45 gpcd range. Also assumed is aggressive I/I control, which reduces I/I to 25 to 100 gpcd. Much higher DWF concentrations will occur.

Thus, future water conservation and I/I control practices can be expected to have a significant impact on wastewater discharges or dry-weather flow. Having to deal with much lower volumes of water opens up opportunities for innovative stormwater management. For example, Pruel (1996) suggests storing DWF on-site during wet-weather periods. If only black water has to be stored, then this option becomes more attractive.

Table 3-15. Final monthly water budget for Denver, CO.

Monthly
(All values are in inches)

Month	Precip-itation	Indoor Water Use	Outdoor Water Use	Total	DWF	I/I	Total	Urban Runoff	Days/month
1	0.5	0.69	0.00	0.69	0.69	0.34	1.03	0.13	31
2	0.7	0.62	0.00	0.62	0.62	0.43	1.05	0.18	28
3	1.3	0.69	0.17	0.86	0.69	0.89	1.58	0.33	31
4	1.5	0.66	0.55	1.22	0.66	0.99	1.66	0.38	30
5	2.0	0.69	1.03	1.71	0.69	1.37	2.06	0.50	31
6	2.5	0.66	1.93	2.60	0.66	1.66	2.32	0.63	30
7	2.4	0.69	2.40	3.08	0.69	1.64	2.33	0.60	31
8	2.2	0.69	2.00	2.68	0.69	1.51	2.19	0.55	31
9	1.2	0.66	0.77	1.44	0.66	0.80	1.46	0.30	30
10	1.0	0.69	0.23	0.91	0.69	0.69	1.37	0.25	31
11	0.7	0.66	0.00	0.66	0.66	0.46	1.13	0.18	30
12	0.6	0.69	0.00	0.69	0.69	0.41	1.10	0.15	31
Total	16.6	8.07	9.08	17.15	8.07	11.19	19.26	4.15	365

Annual
(All values are in inches)

Inputs:			**Quality Aspects**
Precipitation		16.60	High quality
Indoor use		8.07	
Black water	2.29		Could use low quality
Gray water	5.78		Need high quality
Outdoor use		9.08	Need moderate quality
Total		33.75	
Outputs:			
Wastewater			
Legitimate		8.07	Requires high level of treatment
I/I		11.19	Requires modest level of treatment
Urban runoff		4.15	Requires little or no treatment
Roofs	0.83		Requires little or no treatment
Driveways	0.83		Requires little or no treatment
Local streets	1.66		Requires little treatment
Major streets	0.83		Requires moderate treatment
Sub-total, outputs		23.41	
Recharge to local receiving waters and groundwater		10.34	Good quality because of subsurface infiltration
Total		33.74	

Table 3-16. Final monthly water budget for New York, NY.

Monthly
(All values are in inches)

Month	Precip-itation	Indoor Water Use	Outdoor Water Use	Total	DWF	I/I	Total	Urban Runoff	Days/month
1	3.0	0.69	0.00	0.69	0.69	0.69	1.37	0.75	31
2	4.0	0.62	0.00	0.62	0.62	0.82	1.44	1.00	28
3	4.0	0.69	0.00	0.69	0.69	0.91	1.60	1.00	31
4	3.0	0.66	0.00	0.66	0.66	0.66	1.33	0.75	30
5	3.0	0.69	0.46	1.14	0.69	0.69	1.37	0.75	31
6	3.5	0.66	0.77	1.44	0.66	0.77	1.44	0.88	30
7	4.0	0.69	1.14	1.83	0.69	0.91	1.60	1.00	31
8	3.8	0.69	0.80	1.48	0.69	0.87	1.55	0.95	31
9	4.2	0.66	0.33	0.99	0.66	0.93	1.59	1.05	30
10	4.0	0.69	0.00	0.69	0.69	0.91	1.60	1.00	31
11	3.8	0.66	0.00	0.66	0.66	0.84	1.50	0.95	30
12	3.0	0.69	0.00	0.69	0.69	0.69	1.37	0.75	31
Total	43.3	8.07	3.5	11.57	8.07	9.69	17.76	10.83	365

Annual
(All values are in inches)

Inputs:			Quality Aspects
Precipitation		43.30	High quality
Indoor use		8.07	
Black water	2.29		Could use low quality
Gray water	5.78		Need high quality
Outdoor use		3.50	Need moderate quality
Total		54.87	
Outputs:			
Wastewater			
Legitimate		8.07	
Blackwater	2.29		Requires high level of treatment
Graywater	5.78		Requires modest level of treatment
I/I		9.69	Requires little or no treatment
Urban runoff		10.83	
Roofs	2.17		Requires little or no treatment
Driveways	2.17		Requires little or no treatment
Local streets	4.33		Requires little treatment
Major streets	2.17		Requires moderate treatment
Sub-total, outputs		23.41	
Recharge to local receiving waters and groundwater		26.29	Good quality because of subsurface infiltration
Total		54.87	

References

Basta, D.J. and B.T. Bower (Eds.) (1982). Analyzing Natural Systems: Analysis for Regional Residuals-Environmental Quality Management. Resources for the Future, Inc. Washington, D.C.

Cantor, K.P., R. Hoover, P. Hartage et al. (1987). Bladder cancer, drinking water source, and tap water consumption. J. National Cancer Institute. 79(6): 1269-1279.

Clark, R., A. Perkins and S.E. Wood (1997). Water Sustainability in Urban Areas-An Adelaide and Regions Case Study. Report One - An Exploration of the Concept. Dept. of Environment and Natural Resources. Adelaide, Australia. Draft.

Courtney, B.A. (1997). An Integrated Approach to Urban Irrigation: The Role of Shading, Scheduling, and Directly Connected Imperviousness. MS Thesis, Dept. of Civil, Environmental, and Architectural Engineering. U. of Colorado. Boulder, CO.

Denver Water (1997). Comprehensive Annual Financial Report for the Year Ended December 31, 1996. Denver, CO.

DeOreo, W., J. Heaney and P. Mayer (1996). Flow Trace Analysis to Assess Water Use. Journal of the American Water Works Association. Jan., p. 79-80.

Edwards, K. and L. Martin (1995). A Methodology for Surveying Domestic Water Consumption. Journal of the Chartered Institution of Water and Environmental Management. Vol. 9, No. 5., p. 47-489.

Foster, K. E., M. M. Karpiscak and R. G. Brittain (1988a). Casa Del Agua: A Residential Water Conservation and Reuse Demonstration Project in Tucson, AZ. Water Resources Bulletin. 24(6):1201-1206.

Foster, K. E., M. M. Karpiscak and R. G. Brittain (1988b). Casa Del Agua: A Residential Water Conservation and Reuse Demonstration Project in Tucson, AZ. Water Resources Bulletin. 24(6), p. 1201-1206.

Friedler, E., D.M. Brown, and D. Butler (1996). A Study of WC Derived Sewer Solids. Water Science and Technology. Vol. 33, No. 9, p. 17-24.

Fujita, S. (1996). Measures to promote stormwater infiltration. Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. p. 407-412.

Grimmond, C. S. B., T. R. Oke and D. G.Steyn (1986a). Urban Water Balance 1. A Model for Daily Totals. Water Resources Research. 22(10):1397-1403.

Grimmond, C. S. B. and T. R. Oke (1986b). Urban Water Balance 2. Results from a Suburb of Vancouver, British Columbia. Water Resources Research. 22(10):1404-1412.

Grotter, M., and R. Otterpohl (1996). Integrated Urban Water Concept. Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. p. 1801-1806.

Harpring, J. S. (1997). Nature of Indoor Residential Water Use. MS Thesis, Dept. of Civil, Environmental, and Architectural Engineering. U. of Colorado. Boulder, CO.

Heaney, J.P. (1994). Towards Integrated Urban Water System Management. Proc. HEC Workshop on Future Directions of Urban Hydrology. Davis, CA.

Heaney, J.P., L. Wright and Samsuhadi (1996). Risk Analysis in Urban Stormwater Quality Management. in Haimes, Y.Y., Moser, D.A., and E.Z. Stakhiv, (Eds.) Risk-Based Decision Making in Water Resources VII. ASCE. New York, NY. p. 219-248.

Henze, M., L. Somlyody, W. Schilling, and J. Tyson (Eds.) (1997). Sustainable Sanitation. Water Science and Technology. Vol. 35, No. 9.

Herrmann, T. and U. Klaus (1996). Fluxes of nutrients in urban drainage systems: assessment of sources, pathways and treatment technologies. Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. p. 761-766.

Imbe, M., T. Ohta and N. Takano (1996). Quantitative Assessment of Improvement in Hydrological Water Cycle in Urbanized River Basin. Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. p. 1085-1090.

Johnson, H., T.A. Stanstrom, J.S. Svensson, and A. Sandin (1997). Source-Separated Urine-Nutrient and Heavy Metal Contents Water Saving and Fecal Contamination. Water Science and Technology. Vol. 35, No. 9, p. 45-152.

Jones, D. E. (1971). Urban Water Resources Management Affects the Total Urban Picture. Chapter 1F in Albertson, M.L., Tucker, L.S., and Taylor, D.C. (Eds.) Treatise on Urban Water Systems. Colorado State University. Ft. Collins, CO.

Joyce, J. (1995). Odor and Corrosion Control in Collection System: A Growing Problem? Water Environment federation. Sewers of the Future. Alexandria, VA. P.9-1 to 9-12.

Karpiscak, M. M., K. E. Foster and N. Schmidt (1990). Residential Water Conservation: Casa Del Agua. Water Resources Research. 26(6):939-948.

Karpiscak, M.M., R.G Brittain and K.E. Foster (1994). Desert House: A Demonstration/Experiment in Efficient Domestic Water and Energy Use. Water Resources Bulletin. Vol. 30, No. 2, p. 329-334.

Law, I. (1997). Domestic Non-potable Reuse – Why Even Consider it? Water, May/June.

Kneese, A.V., R.U. Ayres and R.C. d'Arge (1970). Economics and the Environment- A Materials Balance Approach. The Johns Hopkins Press. Baltimore, MD.

Maddaus, W.O. (1987). Water Conservation. American Water Works Association. Denver, CO.

Mayer (1995). Residential Water Use and Conservation Effectiveness: A Process Approach. Master's Thesis, University of Colorado. Boulder, CO.

Mayer, P. W., W. B. De Oreo, J. O. Nelson, E. Opitz and R. Allen (1997). North American Residential End Use Study Progress Report to American Water Works Association Research Foundation. Denver, CO.

McPherson, M.B. (1973). Need for Metropolitan Water Balance Inventories. Jour. of the Hydraulics Div. ASCE. 99, HY10, p. 1837-1848.

McPherson, M.B. et al. (1968). Systematic Study and Development of Long-Range Programs of Urban Water Resources Research. Report to Office of Water Resources Research. NTIS No. PB 184 318. Washington, D.C.

Mitchell, V.G., R.G. Mein and T.A. McMahon (1996). Evaluating the Resource Potential of Stormwater and Wastewater: an Australian Perspective. Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. p. 1293-1298.

Nelen, A.J.M., A.C. de Ridder and E.C. Hartman (1996b). Planning of a New Urban Area in a Municipality of Ede. Using a New Approach to Environmental Protection.

Pruell, H.C. (1996). Combined Sewage Prevention System (CSPS) for Domestic Wastewater Source Control. In Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. P. 193-198.

Sakrison, R.G. (1996). New Urbanism, Growth Management and the Effect on Metropolitan Water demands. Proc. of Conserv 96, ASCE, AWRA, and AWWA. Orlando, FL. p. 19-26.

Sieker, F. and H-R. Verworn. (Eds.). Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. p. 259-264.

Stephenson, D. (1996). Evaluation of Effects of Urbanization on Storm Runoff. Proceedings of the 7th International Conference on Urban Storm Drainage. Hannover, Germany. p. 31-36.

Stadjuhar, L. (1997). Outdoor Residential Water Use. Master's Thesis, University of Colorado. Boulder, CO.

US Environmental Protection Agency (USEPA) (1979). 1978 Needs Survey Cost Methodology for Control of Combined Sewer Overflow and Stormwater Discharges. EPA-430/9-79-002. Washington, DC.

Chapter 4

Source Characterization

Robert Pitt

The Source Concept

Urban runoff is comprised of many separate source area flow components that are combined within the drainage area and at the outfall before entering the receiving water. Considering the combined outfall conditions alone may be adequate when evaluating the long term, area-wide effects of many separate outfall discharges to a receiving water. However, if better predictions of outfall characteristics (or the effects of source area controls) are needed, then the separate source area components must be characterized. The discharge at the outfall is made up of a mixture of contributions from different source areas. The "mix" depends on the characteristics of the drainage area and the specific rain event. The effectiveness of source area controls is, therefore, highly site and storm specific.

Various urban source areas all contribute different quantities of runoff and pollutants, depending on their characteristics. Impervious source areas may contribute most of the runoff during small rain events. Examples of these source areas include paved parking lots, streets, driveways, roofs, and sidewalks. Pervious source areas become important contributors for larger rain events. These pervious source areas include gardens, lawns, bare ground, unpaved parking areas and driveways, and undeveloped areas. The relative importance of the individual sources is a function of their areas, their pollutant washoff potentials, and the rain characteristics.

The washoff of debris and soil during a rain is dependent on the energy of the rain and the properties of the material. Pollutants are also removed from source areas by winds, litter pickup, or other cleanup activities. The runoff and pollutants from the source areas flow directly into the drainage system, onto impervious areas that are directly connected to the drainage system, or onto pervious areas that will attenuate some of the flows and pollutants, before they discharge to the drainage system.

Sources of pollutants on paved areas include on-site particulate storage that cannot be removed by usual processes such as rain, wind, and street cleaning. Atmospheric deposition, deposition from activities on these paved surfaces (e.g., auto traffic, material storage) and the erosion of material from upland areas that directly discharge flows onto these areas, are the major sources of pollutants to the paved areas. Pervious areas contribute pollutants mainly through erosion processes where the rain energy dislodges soil from between vegetation. The runoff from these source areas enters the storm drainage system where sedimentation in catchbasins or in the sewerage may affect their ultimate discharge to the outfall. In-stream physical, biological, and chemical processes affect the pollutants after they are discharged to the ultimate receiving water.

Knowing when the different source areas become “active” (when runoff initiates from the area, carrying pollutants to the drainage system) is critical. If pervious source areas are not contributing runoff or pollutants, then the prediction of urban runoff quality is greatly simplified. The mechanisms of washoff and delivery yields of runoff and pollutants from paved areas are much better known than from pervious urban areas (Novotny and Chesters 1981). In many cases, pervious areas are not active except during rain events greater than at least five or ten mm. For smaller rain depths, almost all of the runoff and pollutants originate from impervious surfaces (Pitt 1987). However, in many urban areas, pervious areas may contribute the majority of the runoff, and some pollutants, when rain depths are greater than about 20 mm. The actual importance of the different source areas is highly dependent on the specific land use and rainfall patterns. Obviously, in areas having relatively low-density development, especially where moderate and large sized rains occur frequently (such as in the Southeast), pervious areas typically dominate outfall discharges. In contrast, in areas having significant paved areas, especially where most rains are relatively small (such as in the arid west), the impervious areas dominate outfall discharges. The effectiveness of different source controls is, therefore, quite different for different land uses and climatic patterns.

If the number of events exceeding a water quality objective are important, then the small rain events are of most concern. Stormwater runoff typically exceeds some water quality standards for practically every rain event (especially for bacteria and some heavy metals). In the upper midwest, the median rain depth is about six mm, while in the southeast, the median rain depth is about twice this depth. For these small rain depths and for most urban land uses, directly connected paved areas usually contribute most of the runoff and pollutants. However, if annual mass discharges are more important (e.g. for long-term effects), then the moderate rains are more important. Rains from about 10 to 50 mm produce most of the annual runoff volume in many areas of the U.S. Runoff from both impervious and pervious areas can be very important for these rains. The largest rains (greater than 100 mm) are relatively rare and do not contribute significant amounts of runoff pollutants during normal years, but are very important for drainage design. The specific source areas that are most important (and controllable) for these different conditions vary widely.

This chapter describes sources of urban runoff flows and pollutants based on many studies as found in the literature. This chapter also reports on the specific source area sampling activities conducted as part of this research funded by the USEPA for use in this report.

Sources and Characteristics of Urban Runoff Pollutants

Years of study reveal that the vast majority of stormwater toxicants and much of the conventional pollutants are associated with automobile use and maintenance activities and that these pollutants are strongly associated with the particulates suspended in the stormwater (the non-filterable components or suspended solids). Reducing or modifying automobile use to reduce the use of these compounds, has been difficult with the notable exception of the phasing out of leaded gasoline. Current activities, concentrated in the San Francisco, CA area, focus on encouraging brake pad manufacturers to reduce the use of copper.

The effectiveness of most stormwater control practices is, therefore, dependent on their ability to remove these particles from the water, or possibly from intermediate accumulating locations (such as streets or other surfaces) and not through source reduction. The removal of these particles from stormwater is dependent on various characteristics of these particles, especially their size and settling rates. Some source area controls (most notably street cleaning) affect the particles before they are washed-off and transported by the runoff, while others remove the particles from the flowing water. This discussion, therefore summarizes the accumulation and washoff of these particulates and the particle size distribution of the suspended solids in stormwater runoff to better understand the effectiveness of source area control practices.

Table 4-1 shows that most of the organic compounds found in stormwater are associated with various human-related activities, especially automobile and pesticide use, or are associated with plastics (Verschueren 1983). Heavy metals found in stormwater also mostly originate from automobile use activities, including gasoline combustion, brake lining, fluids (e.g., brake fluid, transmission oil, anti-freeze, grease), undercoatings, and tire wear (Durum 1974, Koeppe 1977, Rubin 1976, Shaheen 1975, Solomon and Natusch 1977, and Wilbur and Hunter 1980). Auto repair, pavement wear, and deicing compound use also contribute heavy metals to stormwater (Field et al. 1973 and Shaheen 1975). Shaheen (1975) found that eroding area soils are the major source of the particulates in stormwater. The eroding area soil particles, and the particles associated with road surface wear, become contaminated with exhaust emissions and runoff containing the polluting compounds. Most of these compounds become tightly bound to these particles and are then transported through the urban area and drainage system, or removed from the stormwater, with the particulates. Stormwater concentrations of zinc, fluoranthene, 1,3-dichlorobenzene, and pyrene are unique in that substantial fractions of these compounds remain in the water and are less associated with the particulates.

All areas are affected by atmospheric deposition, while other sources of pollutants are specific to the activities conducted on the areas. As examples, the ground surfaces of unpaved equipment or material storage areas can become contaminated by spills and debris, while undeveloped land remaining relatively unspoiled by activities can still contribute runoff solids, organics, and nutrients, if eroded. Atmospheric deposition, deposition from activities on paved surfaces, and the erosion of material from upland unconnected areas are the major sources of pollutants in urban areas.

Table 4-1. Uses and sources for organic compounds found in stormwater (Verschueren 1983).

COMPOUND	EXAMPLE USE/SOURCE
Phenol	gasoline, exhaust
N-Nitroso-di-n-propylamine	contaminant of herbicide Treflan
Hexachloroethane	plasticizer in cellulose esters, minor use in rubber and insecticide
Nitrobenzene	solvent, rubber, lubricants
2,4-Dimethylphenol	asphalt, fuel, plastics, pesticides
Hexachlorobutadiene	rubber and polymer solvent, transformer and hydraulic oil
4-Chloro-3-methylphenol	germicide; preservative for glues, gums, inks, textile, and leather
Pentachlorophenol	insecticide, algaecide, herbicide, and fungicide mfg., wood preservative
Fluoranthene	gasoline, motor and lubricating oil, wood preservative
Pyrene	gasoline, asphalt, wood preservative, motor oil
Di-n-octylphthalate	general use of plastics

Many studies have examined different sources of urban runoff pollutants. These references were reviewed as part of this study and the results are summarized in this section. These significant pollutants have been shown to have a potential for creating various receiving water impact problems, as described in Appendix D (???) of this report. Most of these potential problem pollutants typically have significant concentration increases in the urban feeder creeks and sediments, as compared to areas not affected by urban runoff.

The important sources of these pollutants are related to various uses and processes. Automobile related potential sources usually affect road dust and dirt quality more than other particulate components of the runoff system. The road dust and dirt quality is affected by vehicle fluid drips and spills (e.g., gasoline, oils) and vehicle exhaust, along with various vehicle wear, local soil erosion, and pavement wear products. Urban landscaping practices potentially affecting urban runoff include vegetation litter, fertilizer and pesticides. Miscellaneous sources of urban runoff pollutants include firework debris, wildlife and domestic pet wastes and possibly industrial and sanitary wastewaters. Wet and dry atmospheric contributions both affect runoff quality. Pesticide use in an urban area can contribute significant quantities of various toxic materials to urban runoff. Many manufacturing and industrial activities, including the combustion of fuels, also affect urban runoff quality.

Natural weathering and erosion products of rocks contribute the majority of the hardness and iron in urban runoff pollutants. Road dust and associated automobile use activities (gasoline exhaust products) historically contributed most of the lead in urban runoff. However, the decrease of lead in gasoline has resulted in current stormwater lead concentrations being about one tenth of the levels found in stormwater in the early 1970s (Bannerman et al. 1993). In certain situations, paint chipping can also be a major source of lead in urban areas. Road dust, contaminated by tire wear products and zinc plated metal erosion material, contributes most of the zinc to urban runoff. Urban landscaping activities can be a major source of cadmium (Phillips and Russo 1978). Electroplating and ore processing activities can also contribute chromium and cadmium.

Many pollutant sources are specific to a particular area and on-going activities. For example, iron oxides are associated with welding operations and strontium, used in the production of flares and fireworks, would probably be found on the streets in greater quantities around holidays, or at the scenes of traffic accidents. The relative contribution of each of these potential urban runoff sources, is, therefore, highly variable, depending upon specific site conditions and seasons.

Specific information is presented in the following subsections concerning the qualities of various rocks and soils, urban and rural dustfall, and precipitation. This information is presented to assist in the interpretation of the source area runoff samples collected as part of this project.

Chemical Quality of Rocks and Soils

The abundance of common elements in the lithosphere (the earth's crust) is shown in Table 4-2 (Lindsay l979). Almost half of the lithosphere is oxygen and about 25% are silica. Approximately eight percent is aluminum and five percent is iron. Elements comprising between two percent and four percent of the lithosphere include calcium, sodium, potassium and magnesium. Because of the great abundance of these materials in the lithosphere, urban runoff transports only a relatively small portion of these elements to receiving waters, compared to natural processes. Iron and aluminum can both cause detrimental effects in receiving waters if in their dissolved forms. A reduction of the pH substantially increases the abundance of dissolved metals.

Table 4-2. Common elements in the Lithosphere (Lindsay 1979).

Abundance Rank	Element	Concentration in Lithosphere (mg/kg)
1	O	465,000
2	Si	276,000
3	Al	81,000
4	Fe	51,000
5	Ca	36,000
6	Na	28,000
7	K	26,000
8	Mg	21,000
9	P	1,200
10	C	950
11	Mn	900
12	F	625
13	S	600
14	Cl	500
15	Ba	430
16	Rb	280
17	Zr	220
18	Cr	200
19	Sr	150
20	V	150
21	Ni	100

Table 4-3, also from Lindsay (1979), shows the rankings for common elements in soils. These rankings are quite similar to the values shown previously for the lithosphere. Natural soils can contribute pollutants to urban runoff through local erosion. Again, iron and aluminum are very high on this list and receiving water concentrations of these metals are not expected to be significantly affected by urban activities alone.

Table 4-3. Common elements in soils (Lindsay 1979).

Abundance Rank	Element	Typical Minimum (mg/kg)	Typical Maximum (mg/kg)	Typical Average (mg/kg)
1	O	- -	- -	490,000
2	Si	230,000	350,000	320,000
3	Al	10,000	300,000	71,000
4	Fe	7,000	550,000	38,000
5	C	- -	- -	20,000
6	Ca	7,000	500,000	13,700
7	K	400	30,000	8,300
8	Na	750	7,500	6,300
9	Mg	600	6,000	5,000
10	Ti	1,000	10,000	4,000
11	N	200	4,000	1,400
12	S	30	10,000	700
13	Mn	20	3,000	600
14	P	200	5,000	600
15	Ba	100	3,000	430
16	Zr	60	2,000	300
17	F	10	4,000	200
18	Sr	50	1,000	200
19	Cl	20	900	100
20	Cr	1	1,000	100
21	V	20	500	100

The values shown on these tables are expected to vary substantially, depending upon the specific mineral types. Arsenic is mainly concentrated in iron and manganese oxides, shales, clays, sedimentary rocks and phosphorites. Mercury is concentrated mostly in sulfide ores, shales and clays. Lead is fairly uniformly distributed, but can be concentrated in clayey sediments and sulfide deposits. Cadmium can also be concentrated in shales, clays and phosphorites (Durum 1974).

Street Dust and Dirt Pollutant Sources

Characteristics

Most of the street surface dust and dirt materials (by weight) are local soil erosion products, while some materials are contributed by motor vehicle emissions and wear (Shaheen 1975). Minor contributions are made by erosion of street surfaces in good condition. The specific makeup of street surface contaminants is a function of many conditions and varies widely (Pitt 1979).

Automobile tire wear is a major source of zinc in urban runoff and is mostly deposited on street surfaces and nearby adjacent areas. About half of the airborne particulates lost due to tire wear settle out on the street and the majority of the remaining particulates settle within about six meters of the roadway. Exhaust particulates, fluid losses, drips, spills and mechanical wear products can all contribute lead to street dirt. Many heavy metals are important pollutants associated with automobile activity. Most of these automobile pollutants affect parking lots and street surfaces. However, some of the automobile related materials also affect areas adjacent to the streets. This occurs through the wind transport mechanism after being resuspended from the road surface by traffic-induced turbulence.

Automobile exhaust particulates contribute many important heavy metals to street surface particulates and to urban runoff and receiving waters. The most notable of these heavy metals has been lead. However, since the late 1980s, the concentrations of lead in stormwater has decreased substantially (by about ten times) compared to early 1970 observations. This decrease, of course, is associated with significantly decreased consumption of leaded gasoline.

Solomon and Natusch (1977) studied automobile exhaust particulates in conjunction with a comprehensive study of lead in the Champaign-Urbana, IL area. They found that the exhaust particulates existed in two distinct morphological forms. The smallest particulates were almost perfectly spherical, having diameters in the range of 0.1 to 0.5 µm. These small particles consisted almost entirely of PbBrCl (lead, bromine, chlorine) at the time of emission. Because the particles are small, they are expected to remain airborne for considerable distances and can be captured in the lungs when inhaled. The researchers concluded that the small particles are formed by condensation of PbBrCl vapor onto small nucleating centers, which are probably introduced into the engine with the filtered engine air.

Solomon and Natusch (1977) found that the second major form of automobile exhaust particulates were rather large, being roughly 10 to 20 µm in diameter. These particles typically had irregular shapes and somewhat smooth surfaces. The elemental compositions of these irregular particles were found to be quite variable, being predominantly iron, calcium, lead, chlorine and bromine. They found that individual particles did contain aluminum, zinc, sulfur, phosphorus and some carbon, chromium, potassium, sodium, nickel and thallium. Many of these elements (bromine, carbon, chlorine, chromium, potassium, sodium, nickel, phosphorus, lead, sulfur, and thallium) are most likely condensed, or adsorbed, onto the surfaces of these larger particles during passage through the exhaust system. They believed that these large particles originate in the engine or exhaust system because of their very high iron content. They found that 50 to 70 percent of the emitted lead was associated with these large particles, which would be deposited within a few meters of the emission point onto the roadway, because of their aerodynamic properties.

Solomon and Natusch (1977) also examined urban particulates near roadways and homes in urban areas. They found that lead concentrations in soils were higher near roads and houses. This indicated the capability of road dust and peeling house paint to contaminate nearby soils. The lead content of the soils ranged from 130 to about 1,200 mg/kg. Koeppe (1977), during another element of the Champaign-Urbana lead study, found that lead was tightly bound to various soil components. However, the lead did not remain in one location, but it was transported both downward in the soil profile and to adjacent areas through both natural and man-assisted processes.

Street Dirt Accumulation

The washoff of street dirt and the effectiveness of street cleaning as a stormwater control practice are highly dependent on the available street dirt loading. Street dirt loadings are the result of deposition and removal rates, plus "permanent storage." The permanent storage component is a function of street texture and condition and is the quantity of street dust and dirt that cannot be removed naturally or by street cleaning equipment. It is literally trapped in the texture, or cracks, of the street. The street dirt loading at any time is this initial permanent loading plus the accumulation amount corresponding to the exposure period, minus the re-suspended material removal by wind and traffic-induced turbulence. Removal of street dirt can occur naturally by winds and rain, or by human activity (e.g., by the turbulence of traffic or by street cleaning equipment). Very little removal occurs by any process when the street dirt loadings are small, but wind removal may be very large with larger loadings, especially for smooth streets (Pitt 1979).

Figure 4-1 shows very different street dirt loadings for two San Jose, CA residential study areas (Pitt 1979). The accumulation and deposition rates (and therefore the amounts lost to air) are quite similar, but the initial loading values (the permanent storage values) are very different. The loading differences were almost solely caused by the different street textures.

Table 4-4 summarizes many accumulation rate measurements obtained from throughout North America. In the earliest studies (APWA 1969; Sartor and Boyd 1972; and Shaheen 1975), the initial street dirt loading values after a major rain or street cleaning were assumed to be zero. Calculated accumulation rates for rough streets were, therefore, very large. Later tests measured the initial loading values close to the end of major rains and street cleaning and found that they could be very high, depending on the street texture. When these starting loadings were considered, the calculated accumulation rates were, therefore, much lower. The early, uncorrected, Sartor and Boyd accumulation rates that ignored the initial loading values were almost ten times the correct values shown on this table. Unfortunately, most urban stormwater models used these very high early accumulation rates as default values.

The most important factors affecting the initial loading and maximum loading values shown on Table 4-4 were found to be street texture and street condition. When data from many locations are studied, it is apparent that smooth streets have substantially less loadings at any accumulation period compared to rough streets for the same land use. Very long accumulation periods relative to the rain frequency resultant in high street dirt loadings. During these conditions, the wind losses of street dirt (as fugitive dust) may approximate the deposition rate, resulting in relatively constant street dirt loadings. At Bellevue, WA, typical interevent rain periods average about three days. Relatively constant street dirt loadings were observed in Bellevue because the frequent rains kept the loadings low and very close to the initial storage value, with little observed increase in dirt accumulation over time (Pitt 1985). In Castro Valley, CA, the rain interevent periods were much longer (ranging from about 20 to 100 days) and steady loadings were only observed after about 30 days when the loadings became very high and fugitive dust losses caused by the winds and traffic turbulence moderated the loadings (Pitt and Shawley 1982).

An example of the type of research conducted to obtain the values shown in Table 4-4 was conducted by Pitt and McLean (1986) in Toronto. They measured street dirt accumulation rates and the effects of street cleaning as part of a comprehensive stormwater research project. An industrial street with heavy traffic and a residential street with light traffic were monitored about twice a week for three months. At the beginning of this period, intensive street cleaning (one pass per day for each of three consecutive days) was conducted to obtain reasonably clean streets. Street dirt loadings were then monitored every few days to measure the accumulation rates of street dirt. Street dirt sampling procedures developed by Pitt (1979) were applied. Powerful industrial vacuums (two units, each having two HP, combined with a “Y” connector, and using a six inch wide solid aluminum head) were used to clean many separate subsample strips across the roads which were then combined for physical and chemical analyses.

In Toronto, the street dirt particulate loadings were quite high before the initial intensive street cleaning period and were reduced to their lowest observed levels immediately after the last street cleaning. After street cleaning, the loadings on the industrial street increased much faster than for the residential street. Right after intensive cleaning, the street dirt particle sizes were also similar for the two land uses. However, the loadings of larger particles on the industrial street increased at a much faster rate than on the residential street, indicating more erosion or tracking materials being deposited onto the industrial street. The residential street dirt measurements did not indicate that any material was lost to the atmosphere as fugitive dust, probably because of the low street dirt accumulation rate and the short periods of time between rains. The street dirt loadings never had the opportunity to reach the high loading values needed before they could be blown from the streets by winds or by traffic-induced turbulence. The industrial street, in contrast, had a much greater street dirt accumulation rate and reached the critical loading values needed for fugitive losses in the relatively short periods between the rains.

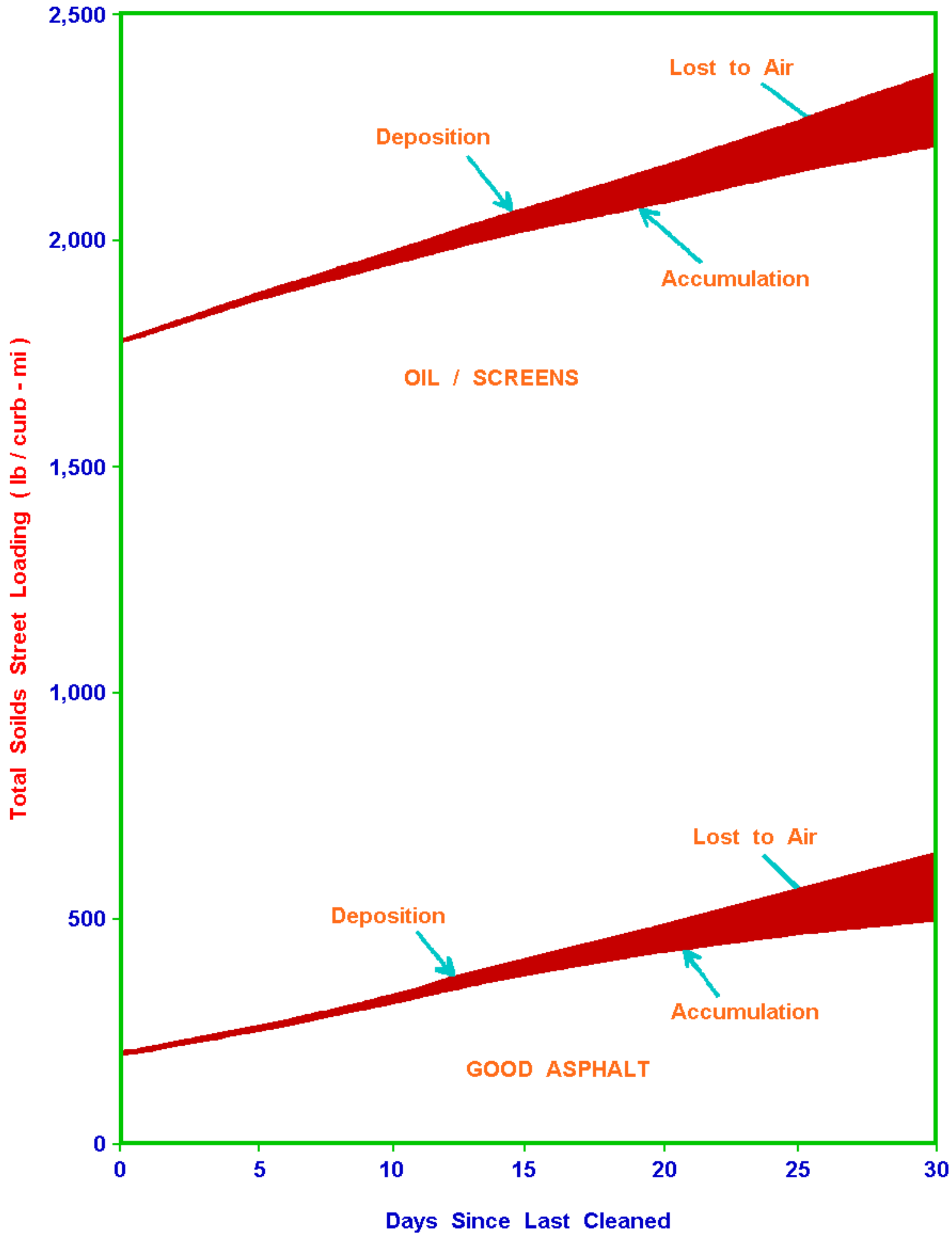


Figure 4-1. Deposition and accumulation of street dirt (Pitt 1979).

Table 4-4. Street dirt loadings and deposition rates.

	Initial Loading Value (grams/curb-meter)	Daily Deposition Rate (grams/curb-meter-day)	Maximum Observed Loading (grams/curb-meter)	Days to Observed Maximum Loading	Reference
Smooth and Intermediate Textured Streets					
Reno/Sparks, NV – good condition	80	1	85	5	Pitt and Sutherland 1982
Reno/Sparks, NV – good with smooth gutters (windy)	250	7	400	30	Pitt and Sutherland 1982
San Jose, CA – good condition	35	4	>140	>50	Pitt 1979
U.S. nationwide – residential streets, good condition	110	6	140	5	Sartor and Boyd 1972 (corrected)
U.S. nationwide – commercial street, good condition	85	4	140	5	Sartor and Boyd 1972 (corrected)
Reno/Sparks, NV – moderate to poor condition	200	2	200	5	Pitt and Sutherland 1982
Reno/Sparks, NV – new residential area (construction)	710	17	910	15	Pitt and Sutherland 1982
Reno/Sparks, NV – poor condition, with lipped gutters	370	15	630	35	Pitt and Sutherland 1982
San Jose, CA – fair to poor condition	80	4	230	70	Pitt 1979
Castro Valley, CA – moderate condition	85	10	290	70	Pitt and Shawley 1982
Ottawa, Ontario – moderate condition	40	20	Na	Na	Pitt 1983
Toronto, Ontario – moderate condition, residential	40	32	100	>10	Pit and McLean 1986
Toronto, Ontario – moderate condition, industrial	60	40	351	>10	Pit and McLean 1986
Believue, WA – dry period, moderate condition	140	6	>230	20	Pitt 1984
Believue, WA – heavy traffic	60	1	110	30	Pitt 1984
Believue, WA – other residential sites	70	3	140	30	Pitt 1984
Average:	150	9	>270	>25	
Range:	35 – 710	1 – 40	85 – 910	5 – 70	
Rough and Very Rough Textured Streets					
San Jose, CA – oil and screens overlay	510	6	>710	>50	Pitt 1979
Ottawa, Ontario – very rough	310	20	Na	Na	Pitt 1983
Reno/Sparks, NV	630	10	860	35	Pitt and Sutherland 1982
Reno/Sparks, NV – windy	540	34	>1,400	>40	Pitt and Sutherland 1982
San Jose, CA – poor condition	220	6	430	30	Pitt 1979
Ottawa, Ontario – rough	200	20	Na	Na	Pitt 1983
U.S. nationwide – industrial streets (poor condition)	190	10	370	10	Sartor and Boyd 1972 (corrected)
Average:	370	15	>750	>30	
Range:	190 - 630	6 - 34	370 - >1,400	10 - >50	

Washoff of Street Dirt

The Yalin equation relates the sediment carrying capacity to runoff flow rate (Yalin 1963). Yalin stated that sediment motion begins when the lift force of flow exceeds a critical lift force. Once a particle is lifted, the drag force of the flow moves it downstream until the weight of the particle forces it back down. The Yalin equation is used to predict particle transport, for specific particle sizes, on a weight per unit flow width basis. It is used for fully turbulent channel flow conditions, typical of shallow overland flow in urban areas. The receding limb (tail) of a hydrograph may have laminar flow conditions, and the suspended sediment carried in the previously turbulent flows would settle out. The predicted constant Yalin sediment load would therefore only occur during periods of rain, and, the sediment load would decrease, due to sedimentation, after the rain stops.

The critical particle bedload tractive force, the tractive force at which the particle begins to move, can be obtained from the Shields' diagram. However, Shen (1981) warned that the Shields' diagram alone cannot be used to predict "self-cleaning" velocities, because it gives only a lower limit below which deposition will occur. It defines the boundary between bed movement and stationary bed conditions. The Shields' diagram does not consider the particulate supply rate in relationship to the particulate transport rate. Reduced particulate transport occurs if the sediment supply rate is less than the transport rate. The Yalin equation by itself is, therefore, not sensitive to particulate supply; it only predicts the carrying capacity of flowing waters.

Besides the particulate supply rate, the Yalin equation is also very sensitive to local flow parameters (specifically gutter flow depth). Therefore, a hydraulic model that can accurately predict sheetflow across impervious surfaces and gutter flow is needed. Sutherland and McCuen (1978) statistically analyzed a modified form of the Yalin equation, in conjunction with a hydraulic model for different gutter flow conditions. Except for the largest particle sizes, the effect of rain intensity on particle washoff was found to be negligible.

The Yalin equation is based on classical sediment transport equations and requires some assumptions concerning the micro-scale aspects of gutter flows and street dirt distributions. The Yalin equation, as typically used in urban stormwater evaluations, assumes that all particles lie within the gutter and no significant washoff occurs by sheetflows traveling across the street towards the gutter. The early measurements of across-the-street dirt distributions made by Sartor and Boyd (1972) indicated that about 90 percent of the street dirt was within about 30 cm of the curb face (typically within the gutter area). These measurements, however, were made in areas of no parking (near fire hydrants because of the need for water for the sampling procedures that were used) and the traffic turbulence was capable of blowing most of the street dirt against the curb barrier (or over the curb onto adjacent sidewalks or landscaped areas) (Shaheen 1975).

In later tests, Pitt (1979) and Pitt and Sutherland (1982) examined street dirt distributions across the street in many additional situations. They found distributions similar to Sartor and Boyd's observations only on smooth streets, with moderate to heavy traffic, and with no on-street parking. In many cases, most of the street dirt was actually in the driving lanes, trapped by the texture of rough streets. If extensive on-street parking was common, much of the street dirt was found on the outside edge of the parking lanes, where much of the resuspended (in air) street dirt blew against the parked cars and settled to the pavement.

Another process that may result in washoff less than predicted by Yalin is bed armoring (Sutherland et al. 1982). As the smaller particulates are removed, the surface is covered by predominantly larger particulates which are not effectively washed-off by rain. Eventually, these larger particulates hinder the washoff of the trapped, underlying, smaller particulates. Debris on the street, especially leaves, can also effectively armor the particulates, reducing the washoff of particulates to very low levels (Singer and Blackard 1978).

Observations of particulate washoff during controlled tests using actual streets and natural street dirt and debris are affected by street dirt distributions and armoring. The earliest controlled street dirt washoff experiments were conducted by Sartor and Boyd (1972) during the summer of 1970 in Bakersfield, CA. Their data were used in many stormwater models (including SWMM, Huber and Heaney 1981; STORM, COE 1975; and HSPF, Donigian and Crawford 1976) to estimate the percentage of the available particulates on the streets that would wash off during rains of different magnitudes. Sartor and Boyd used a rain simulator having many nozzles and a drop height of 1.5 to two meters in street test areas of about five by ten meters. Tests were conducted on concrete, new asphalt, and old asphalt, using simulated rain intensities of about five and 20 mm/hr. They collected and analyzed runoff samples every 15 minutes for about two hours for each test. Sartor and Boyd fitted their data to an exponential curve, assuming that the rate of particle removal of a given size is proportional to the street dirt loading and the constant rain intensity:

$$dN/dt = krN$$

where: dN/dt = the change in street dirt loading per unit time
k = proportionality constant (1/hr)
r = rain intensity (in/hr)
N = street dirt loading (lb/curb-mile)

This equation, upon integration, becomes:

$$N = N_o e^{-krt}$$

where: N = residual street dirt load (after the rain)
N_o = initial street dirt load
t = rain duration (hr)

Street dirt washoff is, therefore, equal to N_o minus N. The variable combination rt, or rain intensity (in/hr) times rain duration (t), is equal to total rain depth (R), in inches. This equation then further reduces to:

$$N = N_o e^{-kR}$$

Therefore, this equation is only sensitive to the total depth of the rain that has fallen since the beginning of the rain, and not rain intensity. Because of decreasing particulate supplies, the exponential washoff curve also predicts decreasing concentrations of particulates with time since the start of a constant rain (Alley 1980 and 1981).

The proportionality constant, k, was found by Sartor and Boyd to be slightly dependent on street texture and condition, but was independent of rain intensity and particle size. The value of this constant is usually taken as 0.18/mm, assuming that 90 percent of the particulates will be washed from a paved surface in one hour during a 13 mm/hr rain. However, Alley (1981) fitted this model to watershed outfall runoff data and found that the constant varied for different storms and pollutants for a single study area. Novotny (as part of Bannerman et al. 1983) also examined "before" and "after" rain event street particulate loading data from the Milwaukee Nationwide Urban Runoff Program (NURP) project and found almost a three-fold difference between the constant value of k for fine (<45 μm) and medium sized particles (100 to 250 μm). The calculated values were 0.026/mm for the fine particles and 0.01/mm for the medium sized particles, both much less than the "accepted" value of 0.18/mm. Jewell et al. (1980) also found large variations in outfall "fitted" constant values for different rains compared to the typical default value. Either the assumption of the high removal of particulates during the 13 mm/hr storm was incorrect or/and the equation cannot be fitted to outfall data (most likely, as this would require that all the particulates are originating from homogeneous paved surfaces during all storm conditions).

This washoff equation has been used in many stormwater models, along with an expression for an availability factor. An availability factor is needed, because N_o is only the portion of the total street load available for washoff. This availability factor (the fraction of the total street dirt loading available for washoff) is generally used as 1.0 for all rain intensities greater than about 18 mm/hr and reduces to about 0.10 for rains of one mm/hr.

The Bellevue, WA urban runoff project (Pitt 1985) included about 50 pairs of street dirt loading observations close to the beginnings and ends of rains. These "before" and "after" loading values were compared to determine significant differences in loadings that may have been caused by the rains. The observations were affected by rains falling directly on the streets, along with flows and particulates originating from non-street areas. The net loading differences were, therefore, affected by street dirt washoff (by direct rains on the street surfaces and by gutter flows augmented by "upstream" area runoff) and by erosion products that originated from non-street areas that may have settled out in the gutters. When all the data were considered together, the net loading difference was about 10 to 13 g/curb-m removed. This amounted to a street dirt load reduction of about 15 percent, which was much less than predicted using either of the two previously described washoff models. Very large reductions in street dirt loadings during rains were observed in Bellevue for the smallest particles, but the largest particles actually increased in loadings (due to deposited erosion materials originating from off-street areas). The particles were not source limited, but armor shielding may have been important. Most of the particulates in the runoff were in the fine particle sizes (<63 μm). Very few particles greater than 1000 μm were found in the washoff water. Care must be taken to not confuse street dirt particle size distributions with stormwater runoff particle size distributions. The stormwater particle size distributions are much more biased towards the smaller sizes, as described later.

Suspended solids washoff predictions for Bellevue conditions were made using the Sutherland and McCuen modification of the Yalin equation and the Sartor and Boyd equation. Three particle size groups (<63, 250-500, and 2000-6350 μm), and three rains, having depths of 5, 10, and 20 mm and 3-hr durations, were considered. The gutter lengths for the Bellevue test areas averaged about 80 m, with gutter slopes of about 4.5%. Typical total initial street dirt loadings for the three particle sizes were: 9 g/curb-m for <63 μm, 18 g/curb-m for 250-500 μm, and 9 g/curb-m for 2000-6350 μm. The actual Bellevue net loading removals during the storms were about 45% for the smallest particle size group, 17% for the middle particle size group, and minus six percent (six percent loading increase) for the largest particle size group. The predicted removals were 90 to 100% using the Sutherland and McCuen method, 61 to 98% using the Sartor and Boyd equation, and 8 to 37% using the availability factor with the Sartor and Boyd equation. The ranges given reflect the different rain volumes and intensities only. There were no large predicted differences in removal percentages as a function of particle size. The availability factor with the Sartor and Boyd equation resulted in the closest predicted values, but the great differences in washoff as a function of particle size was not predicted.

The Bellevue street dirt washoff observations included effects of additional runoff water and particulates originating from non-street areas. The additional flows should have produced more gutter particulate washoff, but upland erosion materials may also have settled in the gutters (as noted for the large particles). However, across-the-street particulate loading measurements indicated that much of the street dirt was in the street lanes, not in the gutters, before and after rains. This particulate distribution reduces the importance of these extra flows and particulates from upland areas. The increased loadings of the largest particles after rains were obviously caused by upland erosion, but the magnitude of the settled amounts was quite small compared to the total street dirt loadings.

In order to clarify street dirt washoff, Pitt (1987) conducted numerous controlled washoff tests on city streets in Toronto. These tests were arranged as an overlapping series of 2^3 factorial tests, and were analyzed using standard factorial test procedures described by Box et al. (1978). The experimental factors examined included: rain intensity, street texture, and street dirt loading. The differences between available and total street dirt loads were also related to the experimental factors. The samples were analyzed for total solids (total residue), dissolved solids (filterable residue: <0.45 µm), and SS (particulate residue: >0.45 µm). Runoff samples were also filtered through 0.45 µm filters and the filters were microscopically analyzed (using low power polarized light microscopes to differentiate between inorganic and organic debris) to determine particulate size distributions from about 1 to 500 µm. The runoff flow quantities were also carefully monitored to determine the magnitude of initial and total rain water losses on impervious surfaces.

The total solids concentrations varied from about 25 to 3000 mg/l, with an obvious decrease in concentrations with increasing rain depths during these constant rain intensity tests. No concentrations greater than 500 mg/l occurred after about two mm of rain. All concentrations after about 10 mm of rain were less than 100 mg/l. Total solids concentrations were independent of the test conditions. A wide range in runoff concentrations was also observed for SS, with concentrations ranging from about 1 to 3000 mg/l. Again, a decreasing trend of concentrations was seen with increasing rain depths, but the data scatter was larger because of the experimental factors. The dissolved solids (<0.45 µm) concentrations ranged from about 20 to 900 mg/l, comprising a surprisingly large percentage of the total solids loadings. For small rain depths, dissolved solids comprised up to 90 percent of the total solids. After 10 mm of rain depth, the filterable residue concentrations were all less than about 50 mg/l.

Manual particle size analyses were also conducted on the suspended solids washoff samples, using a microscope with a calibrated recticle. Figures 4-2 and 4-3 are examples of particle size distributions for two tests. These plots show the percentage of the particles that were less than various sizes, by measured particle volume (assumed to be similar to weight). The plots also indicate median particle sizes of about 10 to 50 µm, depending on when the sample was obtained during the washoff tests. All of the distributions showed surprisingly similar trends of particle sizes with elapsed rain depth. The median size for the sample obtained at about one mm of rain was much greater than for the samples taken after more rain. The median particle sizes of material remaining on the streets after the washoff tests were also much larger than for most of the runoff samples, but were quite close to the initial samples' median particle sizes. The washoff water at the very beginning of the test rains, therefore, contained many more larger particles than during later portions of the rains. Also, a substantial amount of larger particles remained on the streets after the test rains. Most street runoff waters during test rains in the 5 to 15 mm depth category had median suspended solids particle sizes of about 10 to 50 µm. However, dissolved solids (less than 0.45 µm) made up most of the total solids washoff for elapsed rain depths greater than about five mm.

These particle size distributions indicate that the smaller particles were much more important than indicated during previous tests. As an example, the Sartor and Boyd (1972) washoff tests (rain intensities of 50 mm/h for two hour durations) found median particle sizes of about 150 μm which were typically three to five times larger than were found during these tests. They also did not find any significant particle size distribution differences for different rain depths (or rain duration), in contrast to the Toronto tests, which were conducted at more likely rain intensities (3 to 12 mm/hr for two hours).

The particulate washoff values obtained during these Toronto tests were expressed in units of grams per square meter and grams per curb-meter, concentrations (mg/l), and the percent of the total initial loading washed off during the test. Plots of accumulative washoff are shown on Figures 4-4 through 4-11. These plots show the asymptotic washoff values observed in the tests, along with the measured total street dirt loadings. The maximum asymptotic values are the "available" street dirt loadings (N_o). The measured total loadings are seen to be several times larger than these "available" loading values. As an example, the asymptotic available total solids value for the HDS (high intensity rain, dirty street, smooth street) test (Figure 4-10) was about 3 g/m^2 while the total load on the street for this test was about 14 g/m^2, or about five times the available load. The differences between available and total loadings for the other tests were even greater, with the total loads typically about ten times greater than the available loads. The total loading and available loading values for dissolved solids were quite close, indicating almost complete washoff of the very small particles. However, the differences between the two loading values for SS were much greater. Shielding, therefore, may not have been very important during these tests, as almost all of the smallest particles were removed, even in the presence of heavy loadings of large particles.

The actual data are shown on these figures, along with the fitted Sartor and Boyd exponential washoff equations. In many cases, the fitted washoff equations greatly over-predicted suspended solids washoff during the very small rains (usually less than one to three mm in depth). In all cases, the fitted washoff equations described suspended solids washoff very well for rains greater than about 10 mm in depth.

Table 4-5 presents the equation parameters for each of the eight washoff tests for suspended solids. Pitt (1987) concluded that particulate washoff should be divided into two main categories, one for high intensity rains with dirty streets, possibly divided into categories by street texture, and the other for all other conditions. Factorial tests also found that the availability factor (the ratio of the available loading, N_o, to the total loading) varied depending on the rain intensity and the street roughness, as indicated below:

- Low rain intensity and rough streets: 0.045
- High rain intensity and rough streets, or low rain intensity and smooth streets: 0.075
- High rain intensity and smooth streets: 0.20

Obviously, washoff was more efficient for the higher rain energy and smoother pavement tests. The worst case was for a low rain intensity and rough street, where only about 4.5% of the street dirt would be washed from the pavement. In contrast, the high rain intensities on the smooth streets were more than four times more efficient in removing the street dirt.

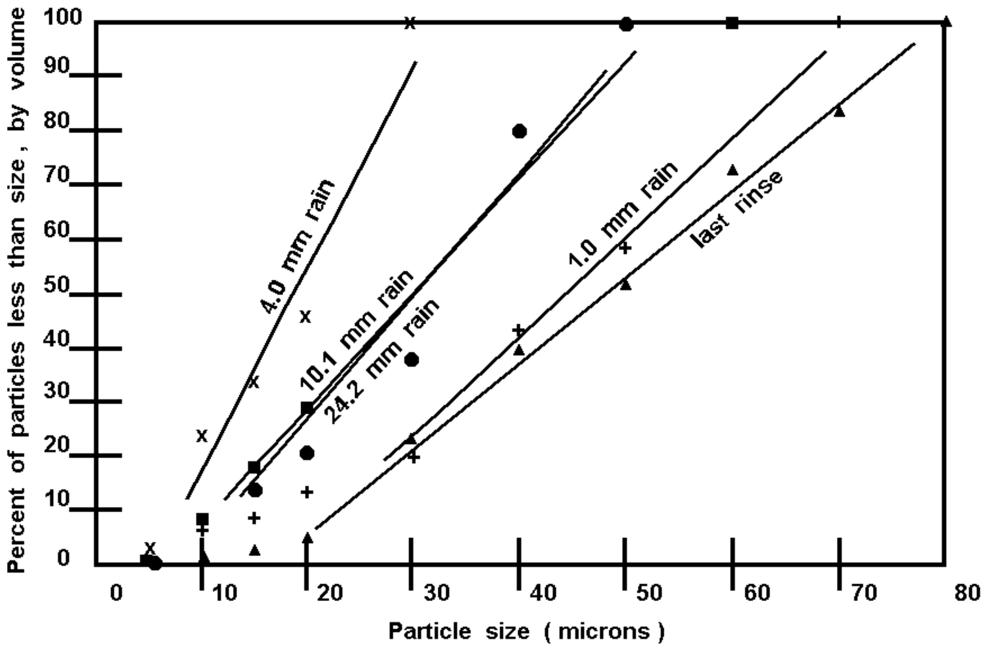


Figure 4-2. Particle size distribution of HDS test (high rain intensity, dirty, and smooth street) (Pitt 1987).

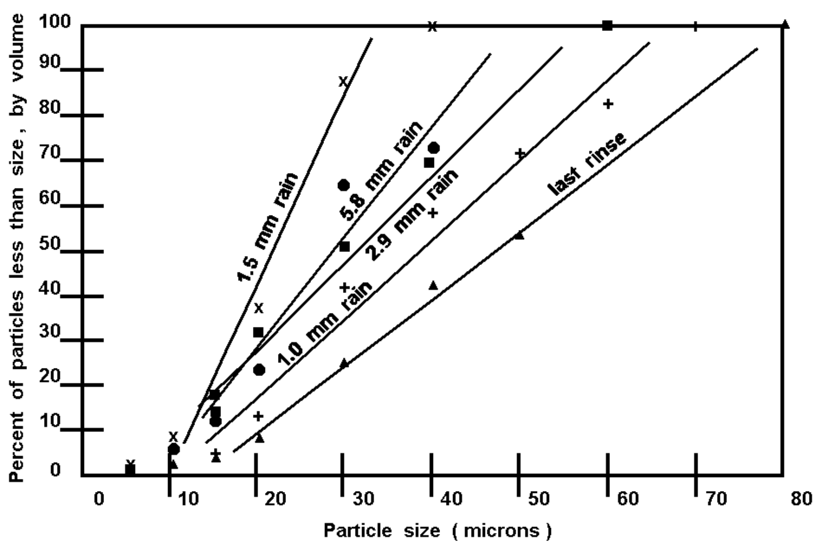


Figure 4-3. Particle size distribution for LCR test (light rain intensity, clean, and rough street) (Pitt 1987).

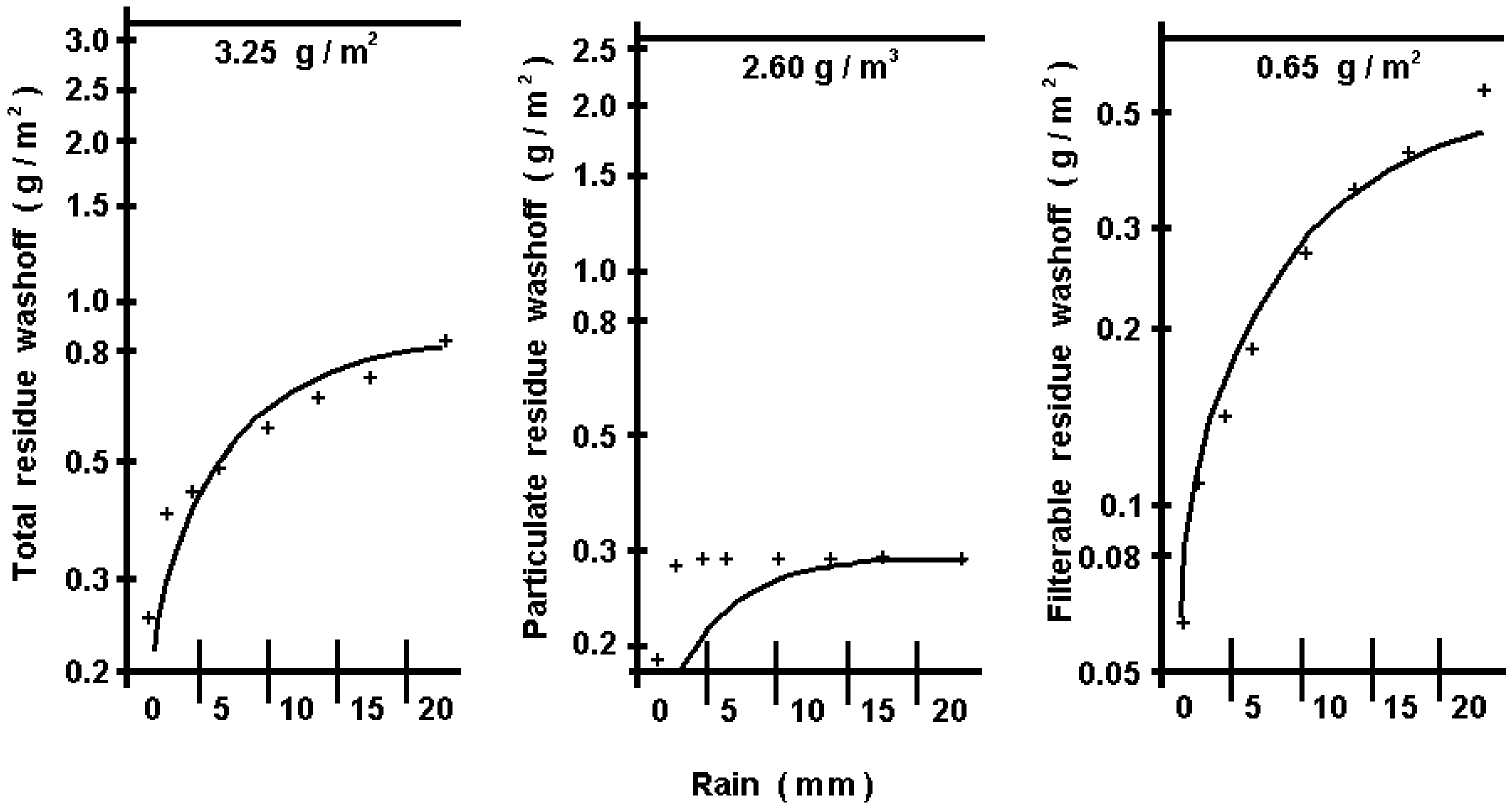


Figure 4-4. Washoff plots for HCR test (high rain intensity, clean, and rough street) (Pitt 1987).

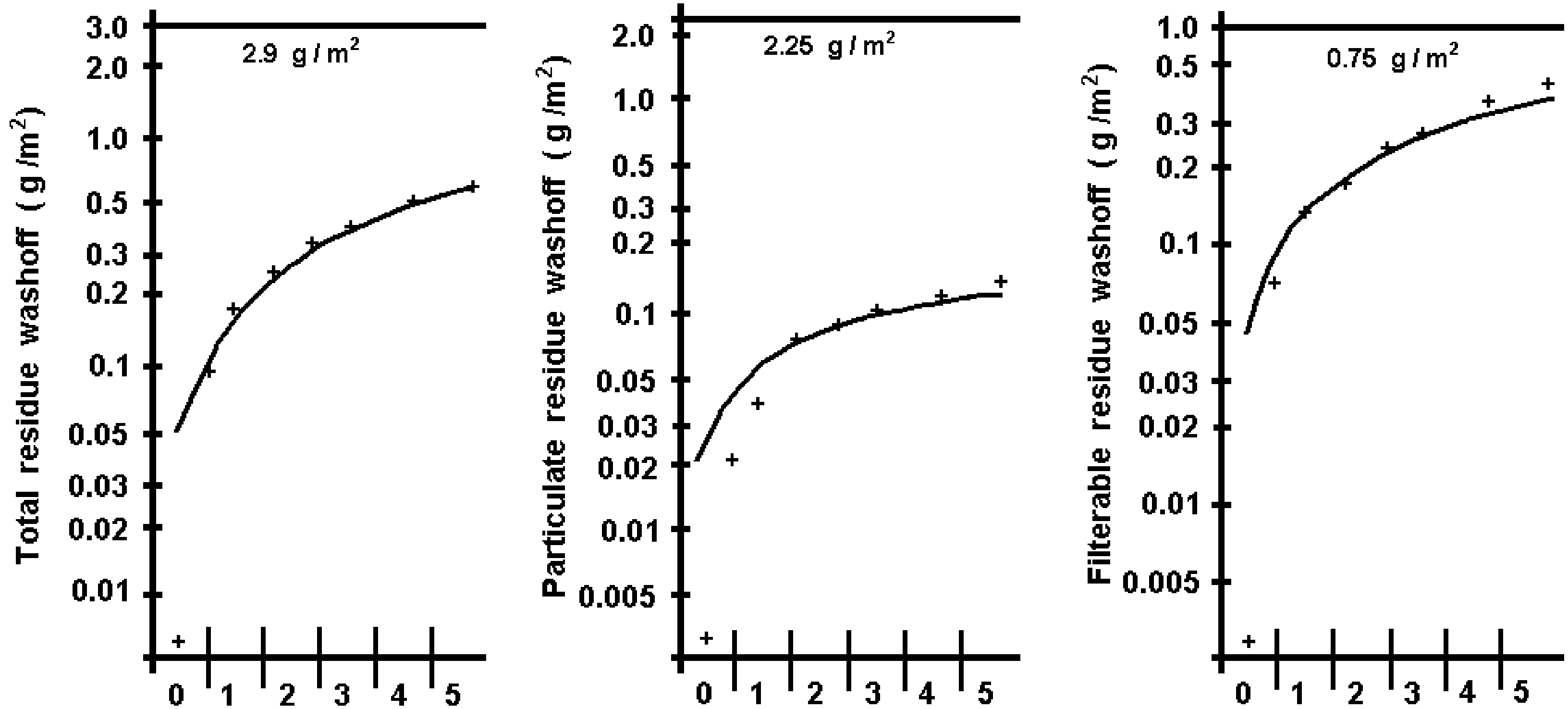


Figure 4-5. Washoff plots for LCR test (light rain intensity, clean, and rough street) (Pitt 1987).

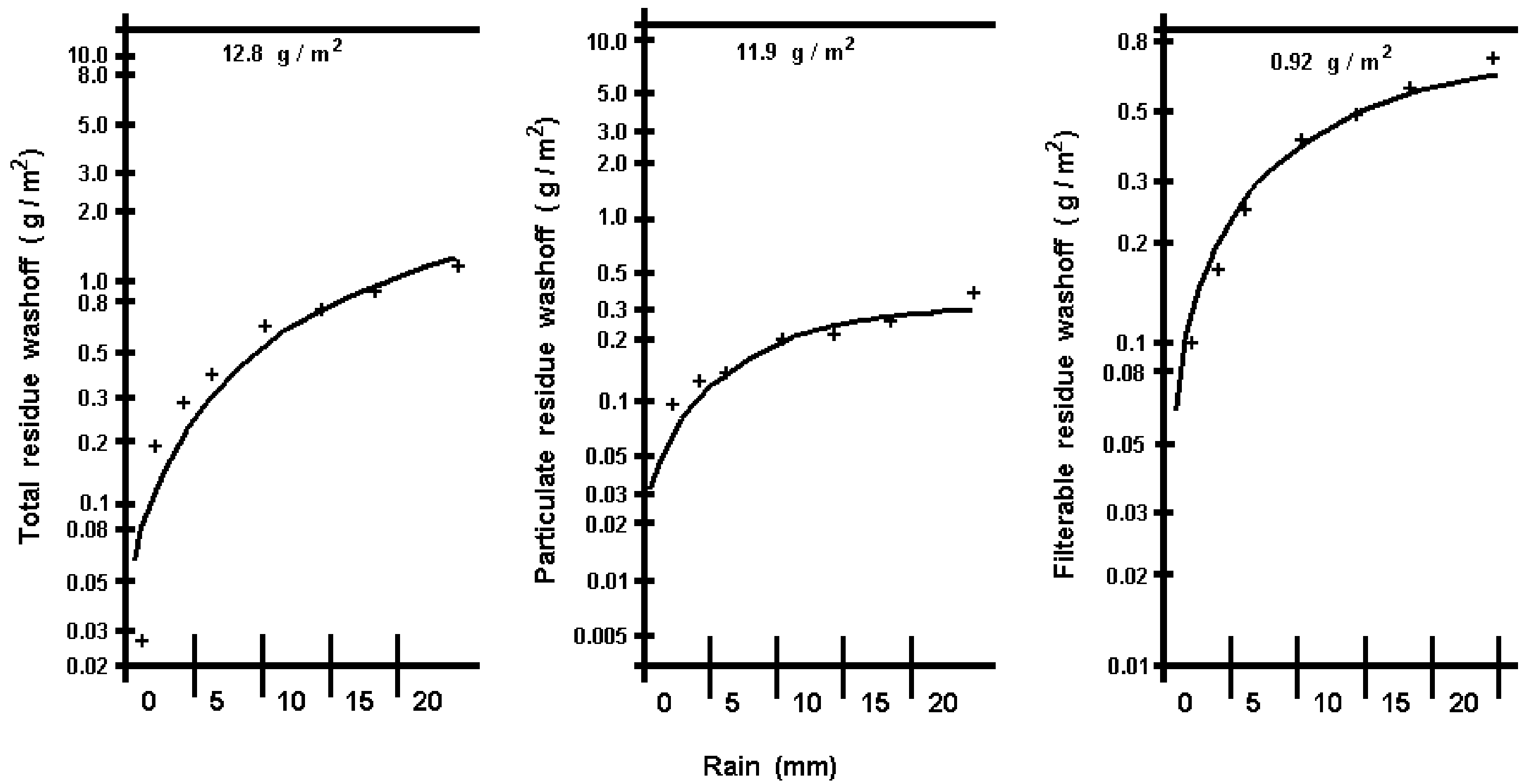


Figure 4-6. Washoff plots for HDR test (high rain intensity, dirty, and rough street) (Pitt 1987).

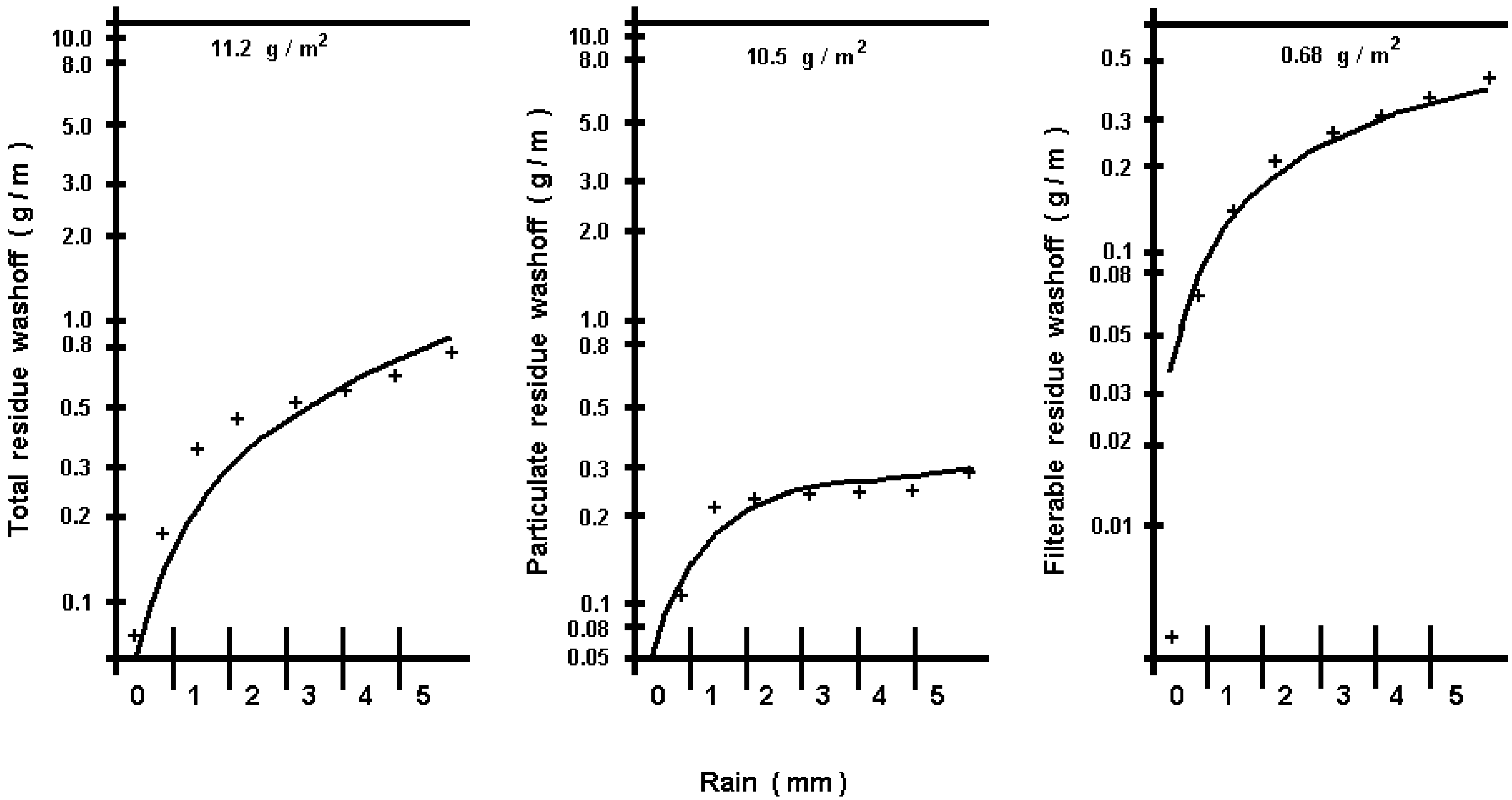


Figure 4-7. Washoff plots for LDR test (light rain intensity, dirty, and rough street) (Pitt 1987).

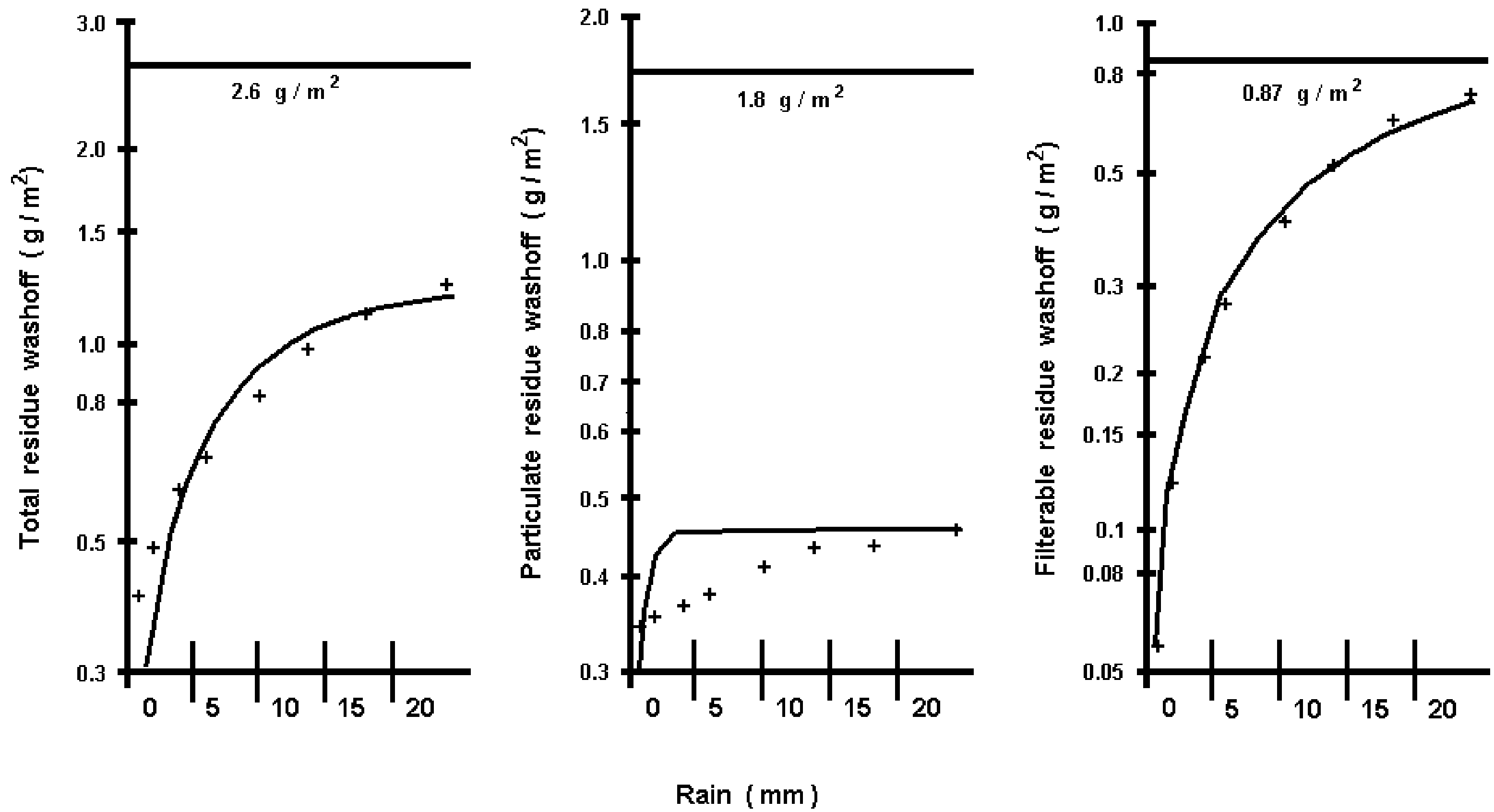


Figure 4-8. Washoff plots for HCS test (high rain intensity, clean, and smooth street) (Pitt 1987).

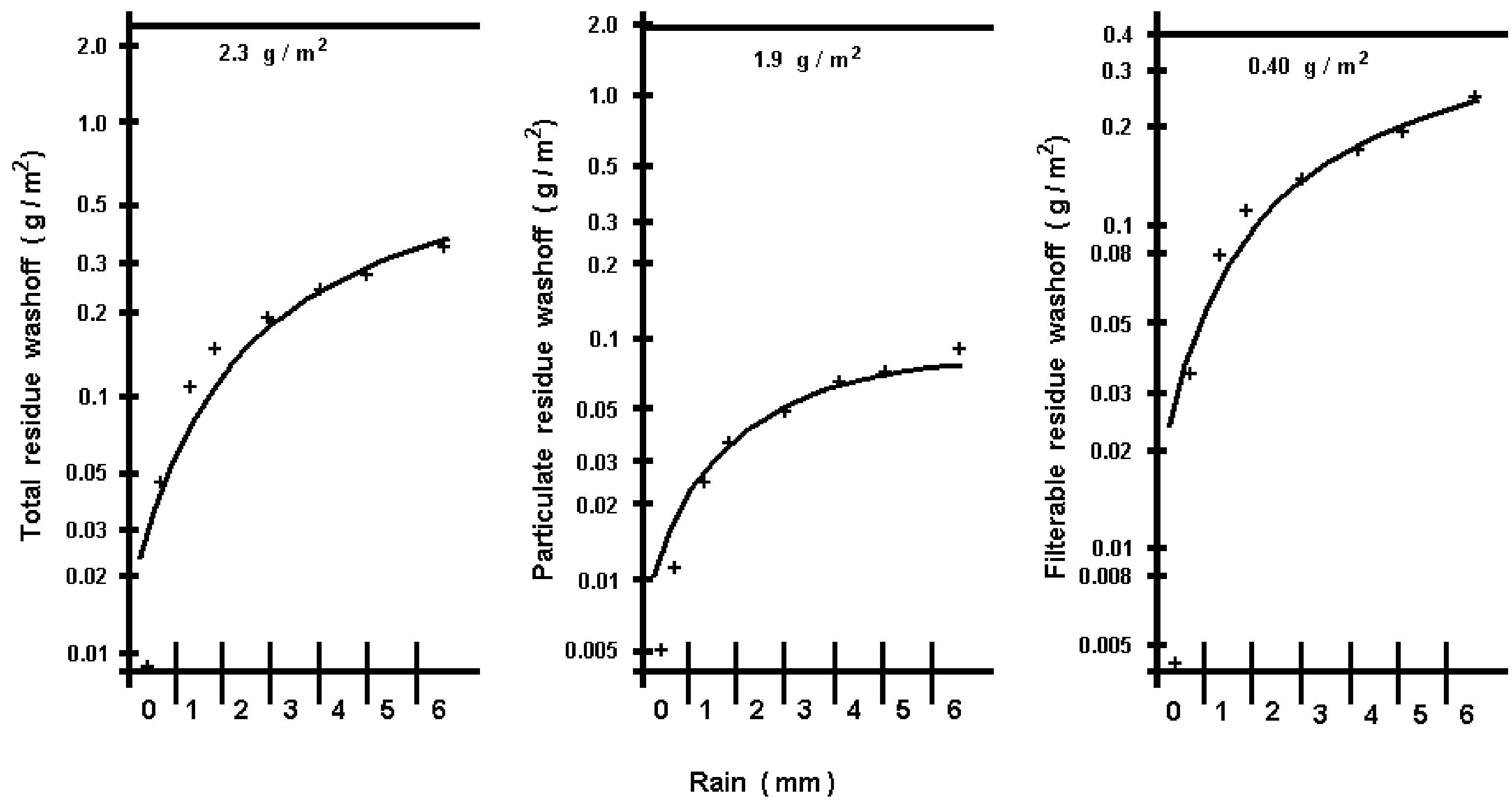


Figure 4-9. Washoff plots for LCS test (light rain intensity, clean, and smooth street) (Pitt 1987).

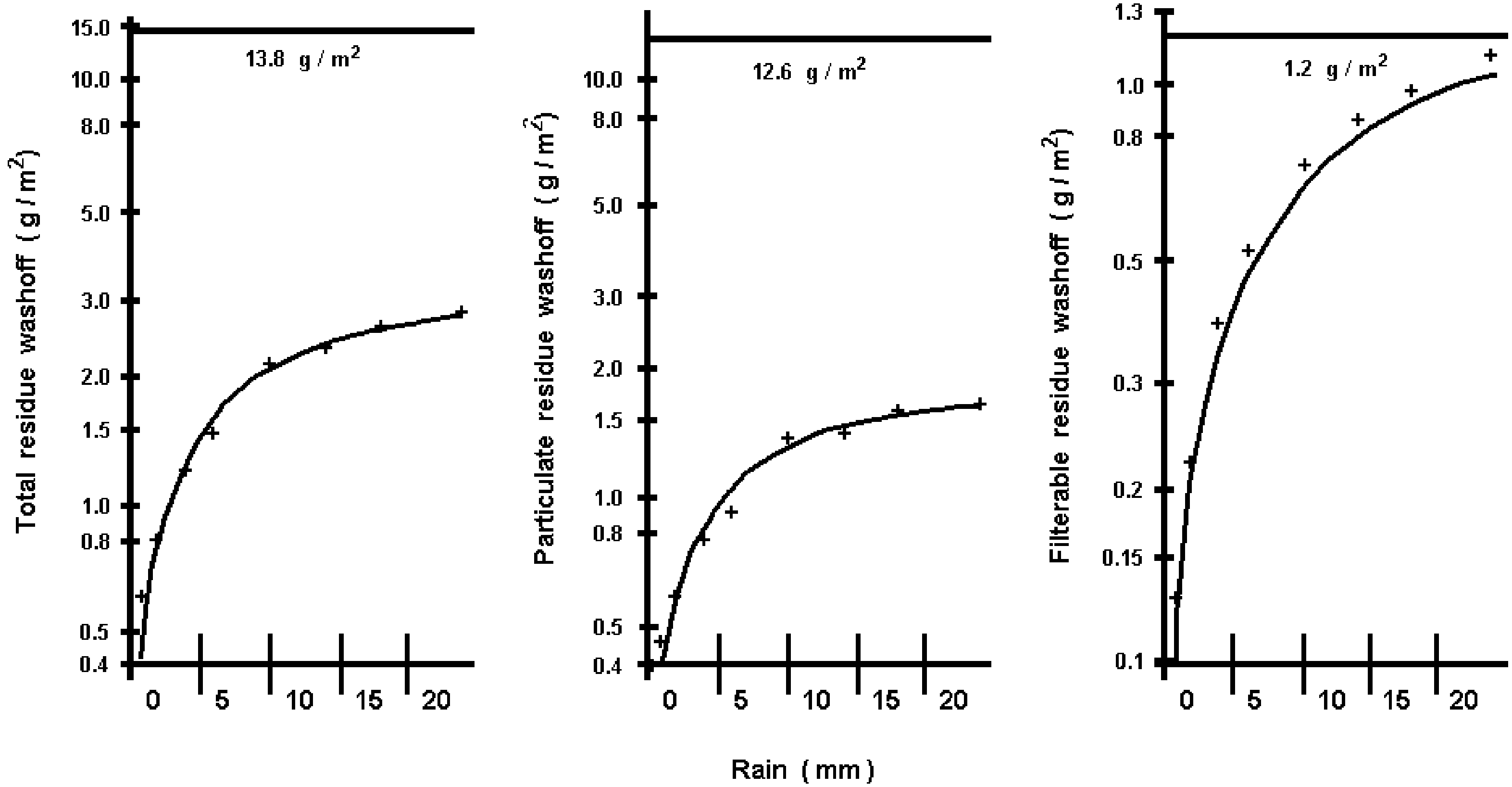


Figure 4-10. Washoff plots for HDS test (high rain intensity, dirty, and smooth street) (Pitt 1987).

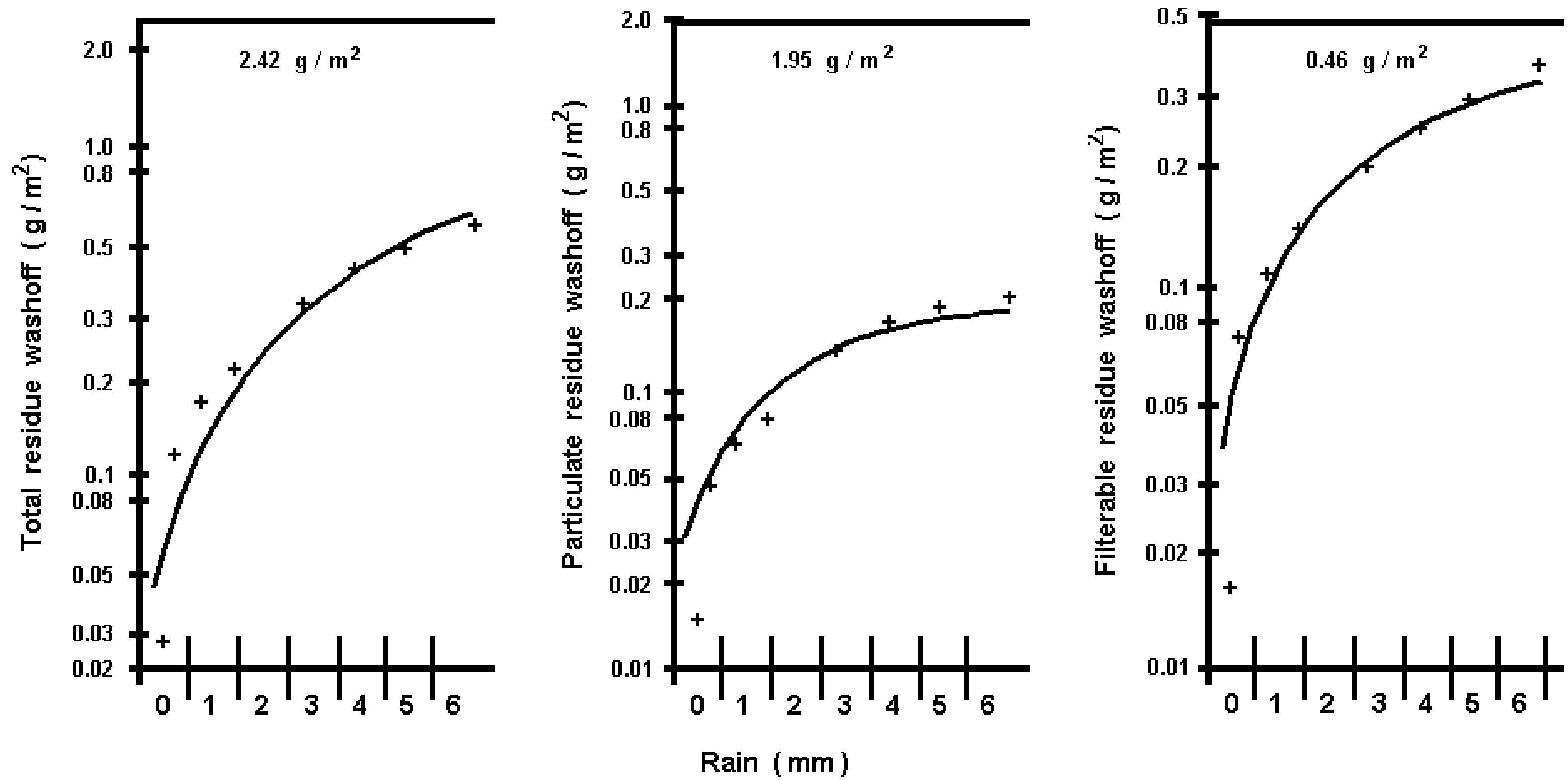


Figure 4-11. Washoff plots for LCS replicate test (light rain intensity, clean, and smooth street) (Pitt 1987).

Table 4-5. Suspended solids washoff coefficients (Pitt 1987)[1].

Test condition code	Rain intensity category	Street dirt loading category	Street texture category	Calculated k (1/hr)	Standard error for k (1/hr)	Ratio of available load to total initial load
HCR	high	clean	rough	0.832	0.064	0.11
LCR	low	clean	rough	0.344	0.038	0.061
HDR	high	dirty	rough	0.077	0.008	0.032
LDR	low	dirty	rough	0.619	0.052	0.028
HCS	high	clean	smooth	1.007	0.321	0.26
LCS	low	clean	smooth	0.302	0.024	0.047
HDS	high	dirty	smooth	0.167	0.015	0.13
LCS	low	clean	smooth	0.335	0.031	0.11

1) Note:

$$N = N_o e^{-kR}$$

where: N = residual street dirt load, after the rain (lb/curb-mile)
N_o = initial street dirt load (lb/curb-mile)
R = rain depth (inches)
k = proportionality constant (1/hr)

Observed Particle Size Distributions in Stormwater

The particle size distributions of stormwater greatly affect the ability of most controls to reduce pollutant discharges. This research included particle size analyses of 121 stormwater samples from three states that were not affected by stormwater controls (southern New Jersey as part of the inlet tests; Birmingham, AL as part of the MCTT pilot-scale tests; and in Milwaukee and Minocqua, WI, as part of the MCTT full-scale tests). These samples represented stormwater entering the stormwater controls being tested. Particle sizes were measured using a Coulter Multi-Sizer IIe and verified with microscopic, sieve, and settling column tests.

Figures 4-12 through 4-14 are grouped box and whisker plots showing the particle sizes (in µm) corresponding to the 10^{th}, 50^{th} (median) and 90^{th} percentiles of the cumulative distributions. If 90% control of SS was desired, for example, then the particles larger than the 90^{th} percentile would have to be removed. The median particle sizes ranged from 0.6 to 38 µm and averaged 14 µm. The 90^{th} percentile sizes ranged from 0.5 to 11 µm and averaged 3 µm. These particle sizes are all substantially smaller than have been typically assumed for stormwater. In all cases, the New Jersey samples had the smallest particle sizes, followed by Wisconsin, and then Birmingham, AL, which had the largest particles. The New Jersey samples were obtained from gutter flows in a residential semi-xeroscaped neighborhood, the Wisconsin samples were obtained from several source areas, including parking areas and gutter flows mostly from residential, but from some commercial areas, and the Birmingham samples were collected from a long-term parking area.

Atmospheric Sources of Urban Runoff Pollutants

Atmospheric processes affecting urban runoff pollutants include dry dustfall and precipitation quality. These have been monitored in many urban and rural areas. In many instances, however, the samples were combined as a bulk precipitation sample before processing. Automatic precipitation sampling equipment can distinguish between dry periods of fallout and precipitation. These devices cover and uncover appropriate collection jars exposed to the atmosphere. Much of this information has been collected as part of the Nationwide Urban Runoff Program (NURP) and the Atmospheric Deposition Program, both sponsored by the USEPA (EPA 1983a).

This information must be interpreted carefully, because of the ability of many polluted dust and dirt particles to be resuspended and then redeposited within the urban area. In many cases, the measured atmospheric deposition measurements include material that was previously residing and measured in other urban runoff pollutant source areas. Also, only small amounts of the atmospheric deposition material would directly contribute to runoff. Rain is subjected to infiltration and the dry fall particulates are likely mostly incorporated with surface soils and only small fractions are then eroded during rains. Therefore, mass balances and determinations of urban runoff deposition and accumulation from different source areas can be highly misleading, unless transfer of material between source areas and the effective yield of this material to the receiving water is considered. Depending on the land use, relatively little of the dustfall in urban areas likely contributes to stormwater discharges.

Dustfall and precipitation affect all of the major urban runoff source areas in an urban area. Dustfall, however, is typically not a major pollutant source but fugitive dust is mostly a mechanism for pollutant transport, as previously mentioned. Most of the dustfall monitored in an urban area is resuspended particulate matter from street surfaces or wind erosion products from vacant areas (Pitt 1979). Point source pollutant emissions can also significantly contribute to dustfall pollution, especially in industrial areas. Transported dust from regional agricultural activities can also significantly affect urban stormwater.

Wind transported materials are commonly called “dustfall.” Dustfall includes sedimentation, coagulation with subsequent sedimentation and impaction. Dustfall is normally measured by collecting dry samples, excluding rainfall and snowfall. If rainout and washout are included, one has a measure of total atmospheric fallout. This total atmospheric fallout is sometimes called “bulk precipitation.” Rainout removes

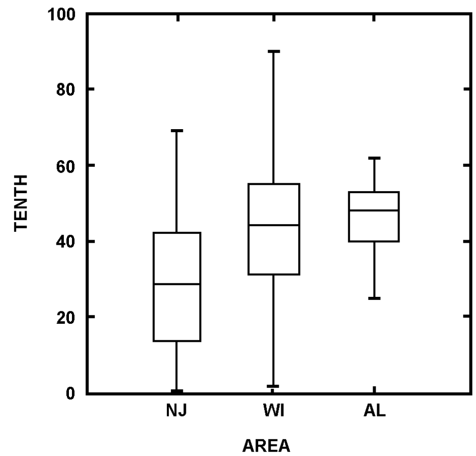


Figure 4-12. Tenth percentile particle sizes for stormwater inlet flows.

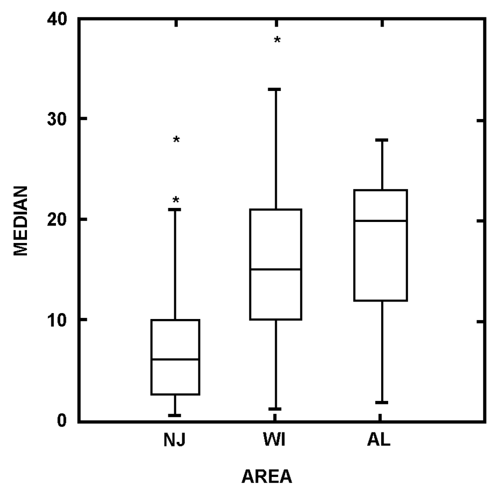


Figure 4-13. Fiftieth percentile particle sizes for stormwater inlet flows.

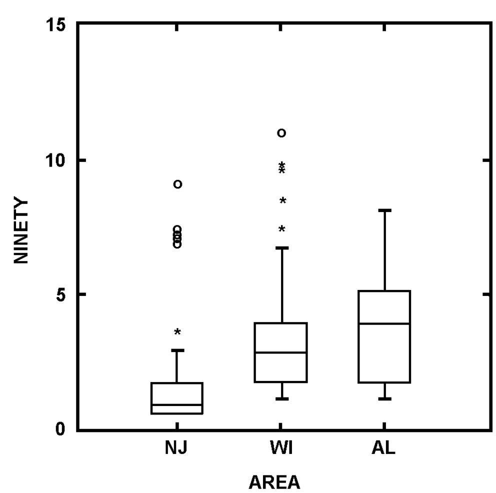


Figure 4-14. Ninetieth percentile particle sizes for stormwater inlet flows.

contaminants from the atmosphere by condensation processes in clouds, while washout is the removal of contaminants by the falling rain. Therefore, precipitation can include natural contamination associated with condensation nuclei in addition to collecting atmospheric pollutants as the rain or snow falls. In some areas, the contaminant contribution by dry deposition is small, compared to the contribution by precipitation (Malmquist 1978). However, in heavily urbanized areas, dustfall can contribute more of an annual load than the wet precipitation, especially when dustfall includes resuspended materials.

Table 4-6 summarizes rain quality reported by several researchers. As expected, the non-urban area rain quality can be substantially better than urban rain quality. Many of the important heavy metals, however, have not been detected in rain in many areas of the country. The most important heavy metals found in rain have been lead and zinc, both being present in rain in concentrations from about 20 μg/l up to several hundred μg/l. It is expected that more recent lead rainfall concentrations would be substantially less, reflecting the decreased use of leaded gasoline since these measurements were taken. Iron is also present in relatively high concentrations in rain (about 30 to 40 μg/l).

Table 4-6. Summary of reported rain quality.

	Rural-Northwest (Quilayute, WA)[1]	Rural-Northeast (Lake George, NY)[1]	Urban-Northwest (Lodi, NJ)[2]	Urban-Midwest (Cincinnati, OH)[3]	Other Urban[3]	Continental Avg. (32 locations)[1]
Suspended solids, mg/l				13		
Volatile suspended solids, mg/l				3.8		
Inorganic nitrogen, mg/l as N				0.69		
Ammonia, mg/l as N					0.7	
Nitrates, mg/l as N					0.3	
Total phosphates, mg/l as P					<0.1	
Ortho phosphate, mg/l as P				0.24		
Scandium, μg/l	<0.002	nd				nd
Titanium, μg/l	nd	nd				nd
Vanadium, μg/l	nd	nd				nd
Chromium, μg/l	<2	nd	1			nd
Manganese, μg/l	2.6	3.4				12
Iron, μg/l	32	35				
Cobalt, μg/l	0.04	nd				nd
Nickel, μg/l	nd	nd	3			43
Copper, μg/l	3.1	8.2	6			21
Zinc, μg/l	20	30	44			107
Lead, μg/l			45			

1) Rubin 1976
2) Wilbur and Hunter 1980
3) Manning et al. 1976

The concentrations of various urban runoff pollutants associated with dry dustfall are summarized in Table 4-7. Urban, rural and oceanic dry dustfall samples contained more than 5,000 mg iron/kg total solids. Zinc and lead were present in high concentrations. These constituents can have concentrations of up to several thousand mg of pollutant per kg of dry dustfall. Spring et al. (1978) monitored dry dustfall near a major freeway in Los Angeles, CA. Based on a series of samples collected over several months, they found that lead concentrations on and near the freeway can be about 3,000 mg/kg, but as low as about 500 mg/kg 150 m (500 feet) away. In contrast, the chromium concentrations of the dustfall did not vary substantially between the two locations and approached oceanic dustfall chromium concentrations.

Table 4-7. Atmosphere dustfall quality.

Constituent, (mg constituent/kg total solids)	Urban[1]	Rural/ suburban[1]	Oceanic[1]	Near freeway (LA)[2]	500' from freeway (LA)[2]
pH				4.3	4.7
Phosphate-Phosphorous				1200	1600
Nitrate-Nitrogen, µg/l				5800	9000
Scandium, µg/l	5	3	4		
Titanium, µg/l	380	810	2700		
Vanadium, µg/l	480	140	18		
Chromium, µg/l	190	270	38	34	45
Manganese, µg/l	6700	1400	1800		
Iron, µg/l	24000	5400	21000		
Cobalt, µg/l	48	27	8		
Nickel, µg/l	950	1400			
Copper, µg/l	1900	2700	4500		
Zinc, µg/l	6700	1400	230		
Lead, µg/l				2800	550

1) Summarized by Rubin 1976
2) Spring 1978

Much of the monitored atmospheric dustfall and precipitation would not reach the urban runoff receiving waters. The percentage of dry atmospheric deposition retained in a rural watershed was extensively monitored and modeled in Oakridge, TN (Barkdoll et al. 1977). They found that about 98% of the lead in dry atmospheric deposits was retained in the watershed, along with about 95% of the cadmium, 85% of the copper, 60% of the chromium and magnesium and 75% of the zinc and mercury. Therefore, if the dry deposition rates were added directly to the yields from other urban runoff pollutant sources, the resultant urban runoff loads would be very much overestimated.

Tables 4-8 and 4-9 report bulk precipitation (dry dustfall plus rainfall) quality and deposition rates as reported by several researchers. For the Knoxville, KY, area (Betson 1978), chemical oxygen demand (COD) was found to be the largest component in the bulk precipitation monitored, followed by filterable residue and nonfilterable residue. Table 4-9 also presents the total watershed bulk precipitation, as the percentage of the total stream flow output, for the three Knoxville watersheds studies. This shows that almost all of the pollutants presented in the urban runoff streamflow outputs could easily be accounted for by bulk precipitation deposition alone. Betson concluded that bulk precipitation is an important component for some of the constituents in urban runoff, but the transport and resuspension of particulates from other areas in the watershed are overriding factors.

Rubin (1976) stated that resuspended urban particulates are returned to the earth's surface and waters in four main ways: gravitational settling, impaction, precipitation and washout. Gravitational settling, as dry deposition, returns most of the particles. This not only involves the settling of relatively large fly ash and soil particles, but also the settling of smaller particles that collide and coagulate. Rubin stated that particles that are less than 0.1 μm in diameter move randomly in the air and collide often with other particles. These small particles can grow rapidly by this coagulation process. These small particles would soon be totally depleted in the air if they were not constantly replenished. Particles in the 0.1 to 1.0 μm range are also removed primarily by coagulation. These larger particles grow more slowly than the smaller particles because they move less rapidly in the air, are somewhat less numerous and, therefore, collide less often with other particles. Particles with diameters larger than 1 μm have appreciable settling velocities. Those particles about 10 μm in diameter can settle rapidly, although they can be kept airborne for extended periods of time and for long distances by atmospheric turbulence.

The second important particulate removal process from the atmosphere is impaction. Impaction of particles near the earth's surface can occur on vegetation, rocks and building surfaces. The third form of particulate removal from the atmosphere is precipitation, in the form of rain and snow. This is caused by the rainout process where the particulates are removed in the cloud-forming process. The fourth important removal process is washout of the particulates below the clouds during the precipitation event. Therefore, it is easy to see that re-entrained particles (especially from street surfaces, other paved surfaces, rooftops and from soil erosion) in urban areas can be readily redeposited through these various processes, either close to the points of origin or at some distance away.

Pitt (1979) monitored airborne concentrations of particulates near typical urban roads. He found that on a number basis, the downwind roadside particulate concentrations were about 10% greater than upwind conditions. About 80% of the concentration increases, by number, were associated with particles in the 0.5 to 1.0 μm size range. However, about 90% of the particle concentration increases by weight were associated with particles greater than 10 μm. Pitt found that the rate of particulate resuspension from street surfaces increases when the streets are dirty (cleaned infrequently) and varied widely for different street and traffic conditions. The resuspension rates were calculated based upon observed long-term accumulation conditions on street surfaces for many different study area conditions, and varied from about 0.30 to 3.6 kg per curb-km (one to 12 lb per curb-mile) of street per day.

Table 4-8. Bulk precipitation quality.

Constituent (all units mg/l except pH)	Urban (average of Knoxville St. Louis & Germany)[1]	Rural (Tennessee)[1]	Urban (Guteburg, Sweden)[2]
Calcium	3.4	0.4	
Magnesium	0.6	0.1	
Sodium	1.2	0.3	
Chlorine	2.5	0.2	
Sulfate	8.0	8.4	
pH	5.0	4.9	
Organic Nitrogen	2.5	1.2	
Ammonia Nitrogen	0.4	0.4	2
Nitrite plus Nitrate-N	0.5	0.4	1
Total phosphate	1.1	0.8	0.03
Potassium	1.8	0.6	
Total iron	0.8	0.7	
Manganese	0.03	0.05	
Lead	0.03	0.01	0.05
Mercury	0.01	0.0002	
Nonfilterable residue	16		
Chemical Oxygen Demand	65		10
Zinc			0.08
Copper			0.02

1) Betson 1978
2) Malmquist 1978

Table 4-9. Urban bulk precipitation deposition rates (Betson 1978)[1].

Rank	Constituent	Average Bulk Deposition Rate (kg/ha/yr)	Average Bulk Prec. as a % of Total Streamflow Output
1	Chemical oxygen demand	530	490
2	Filterable residue	310	60
3	Nonfilterable residue	170	120
4	Alkalinity	150	120
5	Sulfate	96	470
6	Chloride	47	360
7	Calcium	38	170
8	Potassium	21	310
9	Organic nitrogen	17	490
10	Sodium	15	270
11	Silica	11	130
12	Magnesium	9	180
13	Total Phosphate	9	130
14	Nitrite and Nitrate-N	5.7	360
15	Soluble phosphate	5.3	170
16	Ammonia Nitrogen	3.2	1,100
17	Total Iron	1.9	47
18	Fluoride	1.8	300
19	Lead	1.1	650
20	Manganese	0.54	270
21	Arsenic	0.07	720
22	Mercury	0.008	250

1) Average for three Knoxville, KY, watersheds.

Murphy (1975) described a Chicago study where airborne particulate material within the city was microscopically examined, along with street surface particulates. The particulates from both of these areas were found to be similar (mostly limestone and quartz) indicating that the airborne particulates were most likely resuspended street surface particulates, or were from the same source.

PEDCo (1977) found that the re-entrained portion of the traffic-related particulate emissions (by weight) is an order of magnitude greater than the direct emissions accounted for by vehicle exhaust and tire wear. They also found that particulate resuspensions from a street are directly proportional to the traffic volume and that the suspended particulate concentrations near the streets are associated with relatively large particle sizes. The medium particle size found, by weight, was about 15 µm, with about 22% of the particulates occurring at sizes greater than 30 µm. These relatively large particle sizes resulted in substantial particulate fallout near the road. They found that about 15% of the resuspended particulates fall out at 10 m, 25% at 20 m, and 35% at 30 m from the street (by weight).

In a similar study Cowherd et al*.* (1977) reported a wind erosion threshold value of about 5.8 m/s (13 mph). At this wind speed, or greater, significant dust and dirt losses from the road surface could result, even in the absence of traffic-induced turbulence. Rolfe and Reinbold (1977) also found that most of the particulate lead from automobile emissions settled out within 100 m of roads. However, the automobile lead does widely disperse over a large area. They found, through multi-elemental analyses, that the settled outdoor dust collected at or near the curb was contaminated by automobile activity and originated from the streets.

Source Area Sheetflow and Particulate Quality

This section summarizes the source area sheetflow and particulate quality data obtained from several studies conducted in California, Washington, Nevada, Wisconsin, Illinois, Ontario, Colorado, New Hampshire, and New York since 1979. Most of the data obtained were for street dirt chemical quality, but a relatively large amount of parking and roof runoff quality data have also been obtained. Only a few of these studies evaluated a broad range of source areas or land uses.

Source Area Particulate Quality

Particulate potency factors (usually expressed as mg pollutant/kg dry particulate residue) for many samples are summarized on Tables 4-10 and 4-11. These data can help recognize critical source areas, but care must be taken if they are used for predicting runoff quality because of likely differential effects due to washoff and erosion from the different source areas. These data show the variations in chemical quality between particles from different land uses and source areas. Typically, the potency factors increase as the use of an area becomes more intensive, but the variations are slight for different locations throughout the country. Increasing concentrations of heavy metals with decreasing particle sizes was also evident, for those studies that included particle size information. Only the quality of the smallest particle sizes are shown on these tables because they best represent the particles that are removed during rains.

Warm Weather Sheetflow Quality

Sheetflow data, collected during actual rain, are probably more representative of runoff conditions than the previously presented dry particulate quality data because they are not further modified by washoff mechanisms. These data, in conjunction with source area flow quantity information, can be used to predict outfall conditions and the magnitude of the relative sources of critical pollutants. Tables 4-12 through 4-15 summarize warm weather sheetflow observations, separated by source area type and land use, from many locations. The major source area categories are listed below:

1. Roofs
2. Paved parking areas
3. Paved storage areas
4. Unpaved parking and storage areas
5. Paved driveways
6. Unpaved driveways
7. Dirt walks
8. Paved sidewalks
9. Streets
10. Landscaped areas
11. Undeveloped areas
12. Freeway paved lanes and shoulders

Toronto warm weather sheetflow water quality data were plotted against the rain volume that had occurred before the samples were collected to identify any possible trends of concentrations with rain volume (Pitt and McLean 1986). The street runoff data obtained during the special washoff tests reported earlier were also compared with the street sheetflow data obtained during the actual rain events (Pitt 1987). These data observations showed definite trends of solids concentrations verses rain volume for most of the source area categories. Sheetflows from all pervious areas combined had the highest total solids concentrations from any source category, for all rain events. Other paved areas (besides streets) had total solids concentrations similar to runoff from smooth industrial streets. The concentrations of total solids in roof runoff were almost constant for all rain events, being slightly lower for small rains than for large rains. No other pollutant, besides SS, had observed trends of concentrations with rain depths for the samples collected in Toronto. Lead and zinc concentrations were highest in sheetflows from paved parking areas and streets, with some high zinc concentrations also found in roof drainage samples. High bacteria populations were found in sidewalk, road, and some bare ground sheetflow samples (collected from locations where dogs would most likely be "walked").

Some of the Toronto sheetflow contributions were not sufficient to explain the concentrations of some constituents observed in runoff at the outfall. High concentrations of dissolved chromium, dissolved copper, and dissolved zinc in a Toronto industrial outfall during both wet and dry weather could not be explained by wet weather sheetflow observations (Pitt and McLean 1986). As an example, very few detectable chromium observations were obtained in any of the more than 100 surface sheetflow samples analyzed. Similarly, most of the fecal coliform populations observed in sheetflows were significantly lower than those observed at the outfall, especially during snowmelt. It is expected that some industrial wastes, possibly originating from metal plating operations, were the cause of these high concentrations of dissolved metals at the outfall and that some sanitary sewage was entering the storm drainage system.

Table 4-15 summarizes the very little filterable pollutant concentration data available, before this EPA project, for different source areas. Most of the available data are for residential roofs and commercial parking lots.

Table 4-10. Summary of observed street dirt mean chemical quality (mg constituent/kg solids).

Constituent	Residential	Commercial	Industrial
P	620 (4) 540 (6) 1100 (5) 710 (1) 810 (3)	400 (6) 1500 (5) 910 (1)	670 (4)
TKN	1030 (4) 3000 (6) 290 (5) 2630 (3) 3000 (2)	1100 (6) 340 (5) 4300 (2)	560 (4)
COD	100,000 (4) 150,000 (6) 180,000 (5) 280,000 (1) 180,000 (3) 170,000 (2)	110,000 (6) 250,000 (5) 340,000 (1) 210,000 (2)	65,000 (4)
Cu	162 (4) 110 (6) 420 (2)	130 (6) 220 (2)	360 (4)
Pb	1010 (4) 1800 (6) 530 (5) 1200 (1) 1650 (3) 3500 (2)	3500 (6) 2600 (5) 2400 (1) 7500 (2)	900 (4)
Zn	460 (4) 260 (5) 325 (3) 680 (2)	750 (5) 1200 (2)	500 (4)
Cd	<3 (5) 4 (2)	5 (5) 5 (2)	
Cr	42 (4) 31 (5) 170 (2)	65 (5) 180 (2)	70 (4)

References; location; particle size described:

(1) Bannerman et al. 1983 (Milwaukee, WI) <31µm
(2) Pitt 1979 (San Jose, CA) <45 µm
(3) Pitt 1985 (Bellevue, WA) <63 µm
(4) Pitt and McLean 1986 (Toronto, Ontario) <125 µm
(5) Pitt and Sutherland 1982 (Reno/Sparks, NV) <63 µm
(6) Terstriep et al. 1982 (Champaign/Urbana, IL) >63 µm

Table 4-11. Summary of observed particulate quality for other source areas (means for <125 µm particles) (mg constituent/kg solids).

	P	TKN	COD	Cu	Pb	Zn	Cr
Residential/Commercial Land Uses							
	1500	5700	240,000	130	980	1900	77
Roofs	600	790	78,000	145	630	420	47
Paved parking	400	850	50,000	45	160	170	20
Unpaved driveways	550	2750	250,000	170	900	800	70
Paved driveways	360	760	25,000	15	38	50	25
Dirt footpath	1100	3620	146,000	44	1200	430	32
Paved sidewalk	1300	1950	70,000	30	50	120	35
Garden soil	870	720	35,000	35	230	120	25
Road shoulder							
Industrial Land Uses							
Paved parking	770	1060	130,000	1110	650	930	98
Unpaved parking/storage	620	700	110,000	1120	2050	1120	62
Paved footpath	890	1900	120,000	280	460	1300	63
Bare ground	700	1700	70,000	91	135	270	38

Source: Pitt and McLean 1986 (Toronto, Ontario)

Table 4-12. Sheetflow quality summary for other source areas (mean concentration and source of data).

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets
Total Solids (mg/l)									
Residential:	58 (5) 64 (1) 18 (4)	1790 (5)	73 (5)		510 (5)		1240 (5)	49 (5)	325 (5) 235 (4)
Commercial:	95 (1) 190 (4)	340 (2) 240 (1) 102 (7)							325 (4)
Industrial:	113 (5)	490 (5)	270 (5)	1250 (5)	506 (5)	5620 (5)		580 (5)	1800 (5)
Suspended Solids (mg/l)									
Residential:	22 (1) 13 (5)	1660 (5)	41 (5)		440 (5)		810 (5)	20 (5)	242 (5)
Commercial:		270 (2) 65 (1) 41 (7)							242 (5)
Industrial:	4 (5)	306 (5)	202 (5)	730 (5)	373 (5)	4670 (5)		434 (5)	1300 (5)
Dissolved Solids (mg/l)									
Residential:	42 (10 5 (5)	130 (5)	32 (5)		70 (5)		430 (5)	29 (5)	83 (5) 83 (4)
Commercial:		70 (2) 175 (1) 61 (7)							83 (5)
Industrial:	109 (5)	184 (5)	68 (5)	520 (5)	133 (5)	950 (5)		146 (5)	500 (5)

Table 4-12. Sheetflow quality summary for other source areas (mean concentration and source of data) (Continued).

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets
BOD$_5$ (mg/l)									13 (4)
Residential:	3 (4)	22 (4)							
Commercial:	7 (4)	11 (1) 4 (8)							
COD (mg/l)									
Residential:	46 (5) 27 (1) 20 (4)	173 (5)	22 (5)		178 (5)			62 (5)	174 (5) 170 (4)
Commercial:	130 (4)	190 (2) 180 (4) 53 (1) 57 (8)							174 (5)
Industrial:	55 (5)	180 (5)	82 (5)	247 (5)	138 (5)	418 (5)		98 (5)	322 (5)
Total Phosphorus (mg/l)									
Residential:	0.03 (5) 0.05 (1) 0.1 (4)				0.36 (5)		0.20 (5)	0.80 (5)	0.62 (5) 0.31 (4)
Commercial:	0.03 (4) 0.07 (4)	0.16 (1) 0.15 (7) 0.73 (5) 0.9 (2) 0.5 (4)							0.62 (5)
Industrial:	<0.06 (5)	2.3 (5)	0.7 (5)	1.0 (5)	0.9 (5)	3.0 (5)		0.82 (5)	1.6 (5)

Table 4-12. Sheetflow quality summary for other source areas (mean concentration and source of data) (Continued).

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets
Total Phosphate (mg/l)									
Residential:	<0.04 (5) 0.08 (4)				<0.2 (5)		0.66 (5)	0.64 (5)	0.07 (5) 0.12 (4)
Commercial:	0.02 (4)	0.03 (5) 0.3 (2) 0.5 (4) 0.04 (7) 0.22 (8)	<0.02 (5)						0.07 (5)
Industrial:	<0.02 (5)	0.6 (5)	0.06 (5)	0.13 (5)	<0.02 (5)	0.10 (5)		0.03 (5)	0.15 (5)
TKN (mg/l)									
Residential:	1.1 (5) 0.71 (4)				3.1 (5)		1.3 (5)	1.1 (5)	2.4 (5) 2.4 (4)
Commercial:	4.4 (4)	3.8 (5) 4.1 (2) 1.5 (4) 1.0 (1) 0.8 (8)							2.4 (5)
Industrial:	1.7 (5)	2.9 (5)	3.5 (5)	2.7 (5)	5.7 (5)	7.5 (5)		4.7 (5)	5.7 (5)
Ammonia (mg/l)									
Residential:	0.1 (5) 0.9 (1) 0.5 (4)	0.1 (5)	0.3 (5)		<0.1 (5)		0.5 (5)	0.3 (5)	<0.1 (5) 0.42 (4)
Commercial:	1.1 (4)	1.4 (2) 0.35 (4) 0.38 (1)							<0.1 (5)
Industrial:	0.4 (5)	0.3 (5)	0.3 (5)	<0.1 (5)	<0.1 (5)	<0.1 (5)		<0.1 (5)	<0.1 (5)

Table 4-12. Sheetflow quality summary for other source areas (mean concentration and source of data) (Continued).

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets
Phenols (mg/l)									
Residential:	2.4 (5)	12.2 (5)	30.0 (5)		9.7 (5)		<0.4 (5)	8.6 (5)	6.2 (5)
Industrial:	1.2 (5)	9.4 (5)	2.6 (5)	8.7 (5)	7.0 (5)	7.4 (5)		8.7 (5)	24 (7)
Aluminum (µg/l)									
Residential:	0.4 (5)	3.2 (5)	0.38 (5)		5.3 (5)		<0.03 (5)	0.5 (5)	1.5 (5)
Industrial:	<0.2 (5)	3.5 (5)	3.1 (5)	9.2 (5)	3.4 (5)	41 (5)		1.2 (5)	14 (5)
Cadmium (µg/l)									
Residential:	<4 (5) 0.6 (1)	2 (5)	<5 (5)		5 (5)		<1 (5)	<4 (5)	<5 (5)
Commercial:		5.1 (7) 0.6 (8)							<5 (5)
Industrial:	<4 (5)	<4 (5)	<4 (5)	<4 (5)	<4 (5)	<4 (5)		<4 (5)	<4 (5)
Chromium (µg/l)									
Residential:	<60 (5) <5 (4)	20 (5) 71 (4)	<10 (5)		<60 (5)		<10 (5)	<60 (5)	<60 (5) 49 (4)
Commercial:	<5 (4)	19 (7) 12 (8)							<60 (5)
Industrial:	<60 (5)	<60 (5)	<60 (5)	<60 (5)	<60 (5)	70 (5)		<60 (5)	<60 (5)

Table 4-12. Sheetflow quality summary for other source areas (mean concentration and source of data) (Continued).

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets
<u>Copper (µg/l)</u>									
Residential:	10 (5) <5 (4)	100 (5)	20 (5)		210 (5)		20 (5)	20 (5)	40 (5) 30 (4)
Commercial:	110 (4)	40 (2) 46 (4) 110 (7)							40 (5)
Industrial:	<20 (5)	480 (5)	260 (5)	120 (5)	40 (5)	140 (5)		30 (5)	220 (5)
<u>Lead (µg/l)</u>									
Residential:	<40 (5) 30 (3) 48 (1) 17 (4)	250 (5)	760 (5)		1400 (5)		30 (5)	80 (5)	180 (5) 670 (4)
Commercial:	19 (4) 30 (1)	200 (2) 350 (3) 1090 (4) 146 (1) 255 (7) 54 (8)							180 (5)
Industrial:	<40 (5)	230 (5)	280 (5)	210 (5)	260 (5)	340 (5)		<40 (5)	560 (5)

Table 4-12. Sheetflow quality summary for other source areas (mean concentration and source of data) (Continued).

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets
Zinc (µg/l)									
Residential:	320 (5) 670 (1) 180 (4)	520 (5)	390 (5)		1000 (5)		40 (5)	60 (5)	180 (5) 140 (4)
Commercial:	310 (1) 80 (4)	300 (5) 230 (4) 133 (1) 490 (7)							180 (5)
Industrial:	70 (5)	640 (7)	310 (5)	410 (5)	310 (5)	690 (5)		60 (5)	910 (5)

References:

(1) Bannerman et al. 1983 (Milwaukee, WI) (NURP)
(2) Denver Regional Council of Governments 1983 (NURP)
(3) Pitt 1983 (Ottawa)
(4) Pitt and Bozeman 1982 (San Jose)
(5) Pitt and McLean 1986 (Toronto)
(7) STORET Site #590866-2954309 (Shop-Save-Durham, NH) (NURP)
(8) STORET Site #596296-2954843 (Huntington-Long Island, NY) (NURP)

Table 4-13. Sheetflow quality summary for undeveloped landscaped and freeway pavement areas (mean observed concentrations and source of data).

Pollutants	Landscaped Areas	Undeveloped Areas	Freeway Paved Lane and Shoulder Areas
Total Solids, mg/l	388 (4)	588 (4)	340 (5)
Suspended Solids, mg/l	100 (4)	400 (1) 390 (4)	180 (5)
Dissolved Solids, mg/l	288 (4)	193 (4)	160 (5)
BOD_5, mg/l	3 (3)	- - - -	10 (5)
COD, mg/l	70 (3) 26 (4)	72 (1) 54 (4)	130 (5)
Total Phosphorus, mg/l	0.42 (3) 0.56 (4)	0.40 (1) 0.68 (4)	- - - -
Total Phosphate, mg/l	0.32 (3) 0.14 (4)	0.10 (1) 0.26 (4)	0.38 (5)
TKN, mg/l	1.32 (3) 3.6 (4)	2.9 (1) 1.8 (4)	2.5 (5)
Ammonia, mg/l	1.2 (3) 0.4 (4)	0.1 (1) <0.1 (4)	- - - -
Phenols, µg/l	0.8 (4)	- - - -	- - - -
Aluminum, µg/l	1.5 (4)	11 (4)	- - - -
Cadmium, µg/l	<3 (4)	<4 (4)	60 (5)
Chromium, µg/l	10 (3)	<60 (4)	70 (5)
Copper, µg/l	<20 (4)	40 (1) 31 (3) <20 (4)	120 (5)
Lead, µg/l	30 (2) 35 (3) <30 (4)	100 (1) 30 (2) <40 (4)	2000 (5)
Zinc, µg/l	10 (3)	100 (1) 100 (4)	460 (5)

References:
(1) Denver Regional Council of Governments 1983 (NURP)
(2) Pitt 1983 (Ottawa)
(3) Pitt and Bozeman 1982 (San Jose)
(4) Pitt and McLean 1986 (Toronto)
(5) Shelly and Gaboury 1986 (Milwaukee)

Table 4-14. Source area bacteria sheetflow quality summary (means).

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/ Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets	Land-scaped	Un-developed	Freeway Paved Lane and Shoulders
Fecal Coliforms (#/100 ml)												
Residential:	85 (2) <2 (3) 1400 (4)	250,000 (4)	100 (4)		600 (4)			11,000 (4)	920 (3) 6,900 (4)	3300 (4)	5400 (2) 49 (3)	1500 (7)
Commercial	9 (3)	2900 (2) 350 (3) 210 (1) 480 (5) 23,000 (6)										
Industrial:	1600 (4)	8660 (6)	9200 (4)	18,000 (4)	66,000 (4)	300,000 (4)		55,000 (4)	100,000 (4)			
Fecal Strep (#/100 ml)												
Residential:	170 (2) 920 (3) 2200 (4)	190,000 (4)	<100 (4)		1900 (4)		1800 (4)		>2400 (3) 7300 (4)	43,000 (4)	16,500 (2) 920 (3)	2200 (7)
Commercial:	17 (2)	11,900 (2) >2400 (3) 770 (1) 1120 (5) 62,000 (6)										
Industrial:	690 (4)	7300 (4)	2070 (4)	8100 (4)	36,000 (4)	21,000 (4)		3600 (4)	45,000 (4)			
Pseudo, Aerug (#/100 ml)												
Residential:	30,000 (4) 50 (4)	1900 (4)	100 (4)		600 (4)		600 (4)		570 (4)	2100 (4)		
Industrial:		5800 (4)	5850 (4)	14,000 (4)	14,300 (4)	100 (4)		3600 (4)	6200 (4)			

References:

(1) Bannerman et al. 1983 (Milwaukee, WI) (NURP)
(2) Pitt 1983 (Ottawa)
(3) Pitt and Bozeman 1982 (San Jose)
(4) Pitt and McLean 1986 (Toronto)
(5) STORET Site #590866-2954309 (Shop-Save-Durham, NH) (NURP)
(6) STORET Site #596296-2954843 (Huntington-Long Island, NY) (NURP)
(7) Kobriger et al. 1981 and Gupta et al. 1977

Table 4-15. Source area filterable pollutant concentration summary (means).

	Residential			Commercial			Industrial		
	Total	Filterable	Filterable (%)	Total	Filterable	Filterable (%)	Total	Filterable	Filt. (%)
Roof Runoff									
Solids (mg/l)	64 58	42 45	66 (1) 77 (3)				113	110	97 (3)
Phosphorus (mg/l)	0.054	0.013	24 (1)						
Lead (µg/l)	48	4	8 (1)						
Paved Parking									
Solids (mg/l)				240 102 1790	175 61 138	73 (1) 60 (4) 8 (3)	490	138	28 (3)
Phosphorus (mg/l)				0.16 0.9	0.03 0.3	19 (1) 33 (2)			
TKN (mg/l)				0.77	0.48	62 (5)			
Lead (µg/l)				146 54	5 8.8	3 (1) 16 (5)			
Arsenic (µg/l)				0.38	0.095	25 (5)			
Cadmium (µg/l)				0.62	0.11	18 (5)			
Chromium (µg/l)				11.8	2.8	24 (5)			
Paved Storage									
Solids (mg/l)				73	32	44 (3)	270	64	24 (3)

References:

(1) Bannerman et al. 1983 (Milwaukee) (NURP)
(2) Denver Regional Council of Governments 1983 (NURP)
(3) Pitt and McLean 1986 (Toronto)
(4) STORET Site #590866-2954309 (Shop-Save-Durham, NH) (NURP)
(6) STORET Site #596296-2954843 (Huntington-Long Island, NY) (NURP)

Other Pollutant Contributions to the Storm Drainage System

The detection of pentachlophenols in the relatively few samples previously analyzed indicated important leaching from treated wood. Frequent detections of polycyclic aromatic hydrocarbons (PAHs) during the U.S. Environmental Protection Agency's Nationwide Urban Runoff Program (EPA 1983a) may possibly indicate leaching from creosote treated wood, in addition to fossil fuel combustion sources. High concentrations of copper, and some chromium and arsenic observations also indicate the potential of leaching from "CCA" (copper, chromium, and arsenic) treated wood.

The significance of these leachate products in the receiving waters is currently unknown, but alternatives to these preservatives should be considered. Many cities use aluminum and concrete utility poles instead of treated wood poles. This is especially important considering that utility poles are usually located very close to the drainage system ensuring an efficient delivery of leachate products. Many homes currently use wood stains containing pentachlorophenol and other wood preservatives. Similarly, the construction of retaining walls, wood decks and playground equipment with treated wood is common. Some preservatives (especially creosote) cause direct skin irritation, besides contributing to potential problems in receiving waters. Many of these wood products are at least located some distance from the storm drainage system, allowing some improvement to surface water quality by infiltration through pervious surfaces.

Sources of Stormwater Toxicants

This project included the collection and analysis of 87 urban stormwater runoff samples from a variety of source areas under different rain conditions as summarized in Table 4-16. All of the samples were analyzed in filtered (0.45 µm filter) and non-filtered forms to enable partitioning of the toxicants into "particulate" (non-filterable) and "dissolved" (filterable) forms.

Table 4-16. Numbers of samples collected from each source area type.

Local Source Areas [1]	Residential	Commercial/ Institutional	Industrial	Mixed
Roofs	5	3	4	
Parking Areas	2	11	3	
Storage Areas	na	2	6	
Streets	1	1	4	
Loading Docks	na	na	3	
Vehicle Service Area	na	5	na	
Landscaped Areas	2	2	2	
Urban Creeks				19
Detention Ponds				12

1) All collected in Birmingham, AL.

Analyses and Sampling

The samples listed in Table 4-16 were all obtained from the Birmingham, AL, area. Samples were taken from shallow flows originating from homogeneous source areas by using several manual grab sampling procedures. For deep flows, samples were collected directly into the sample bottles. For shallow flows, a peristaltic hand operated vacuum pump created a small vacuum in the sample bottle, which then gently drew the sample directly into the container through a Teflon™ tube. About one liter of sample was needed, split into two containers: one 500 ml glass bottle with Teflon™ lined lid was used for the organic and toxicity analyses and another 500 ml polyethylene bottle was used for the metal and other analyses.

An important aspect of the research was to evaluate the effects of different land uses and source areas, plus the effects of rain characteristics, on sample toxicant concentrations. Therefore, careful records were obtained of the amount of rain and the rain intensity that occurred before the samples were obtained. Antecedent dry period data were also obtained to compare with the chemical data in a series of statistical tests.

All samples were handled, preserved, and analyzed according to accepted protocols (EPA 1982 and 1983b). The organic pollutants were analyzed using two gas chromatographs, one with a mass selective detector (GC/MSD) and another with an electron capture detector (GC/ECD). The pesticides were analyzed according to EPA method 505, while the base neutral compounds were analyzed according to EPA method 625 (but only using 100 ml samples). The pesticides were analyzed on a Perkin Elmer Sigma 300 GC/ECD using a J&W DB-1 capillary column (30m by 0.32 mm ID with a 1 μm film thickness). The base neutrals were analyzed on a Hewlett Packard 5890 GC with a 5970 MSD using a Supelco DB-5 capillary column (30m by 0.25 mm ID with a 0.2 μm film thickness). Table 4-17 lists the organic toxicants that were analyzed.

Metallic toxicants, also listed in Table 4-17, were analyzed using a graphite furnace equipped atomic absorption spectrophotometer (GFAA). EPA methods 202.2 (Al), 213.2 (Cd), 218.2 (Cr), 220.2 (Cu), 239.2 (Pb), 249.2 (Ni), and 289.2 (Zn) were followed in these analyses. A Perkin Elmer 3030B atomic absorption spectrophotometer was used after nitric acid digestion of the samples. Previous research (Pitt and McLean 1986; EPA 1983a) indicated that low detection limits were necessary in order to measure the filtered sample concentrations of the metals, which would not be achieved by use of a standard flame atomic absorption spectrophotometer. Low detection limits would enable partitioning of the metals between the solid and liquid phases to be investigated, an important factor in assessing the fates of the metals in receiving waters and in treatment processes.

Table 4-17. Toxic pollutants analyzed in samples.

Pesticides Detention Limit = 0.3 µg/l	Phthalate Esters Detention Limit = 0.5 µg/l	Polycyclic Aromatic Hydrocarbons Detention Limit = 0.5 µg/l		Metals Detention Limit = 1 µg/l
BHC (Benzene hexachloride)	Bis(2-ethylhexyl) Phthalate	Acenaphthene	Fluoranthene	Aluminum
Heptachlor	Butyl benzyl phthalate	Acenapthylene	Fluorene	Cadmium
Aldrin	Di-n-butyl phthalate	Anthracene	Indeno (1,2,3-cd) pyrene	Chromium
Endosulfan	Diethyl phthalate	Benzo (a) anthracene	Naphthalene	Copper
Heptachlor epoxide	Dimethyl phthalate	Benzo (a) pyrene	Phenanthrene	Lead
DDE (Dichlorodiphenyl dichloroethylene)	Di-n-octyl phthalate	Benzo (b) fluoranthene	Pyrene	Nickel
DDD (Dichlorodiphenyl dichloroethane)		Benzo (ghi) perylene		Zinc
DDT (Dichlorodiphenyl trichloroethane)		Benzo (k) fluoranthene		
Endrin		Chrysene		
Chlordane		Dibenzo (a,h) anthracene		

The Microtox™ 100% sample toxicity screening test, from Azur Environmental (previously Microbics, Inc.), was selected for this research after comparisons with other laboratory bioassay tests. During the first research, 20 source area stormwater samples and combined sewer samples (obtained during a cooperative study being conducted in New York City) were split and sent to four laboratories for analyses using 14 different bioassay tests. Conventional bioassay tests were conducted using freshwater organisms at the EPA's Duluth, MN, laboratory and using marine organisms at the EPA's Narraganssett Bay, RI, laboratory. In addition, other bioassay tests, using bacteria, were also conducted at the Environmental Health Sciences Laboratory at Wright State University, Dayton, OH. The tests represented a range of organisms that included fish, invertebrates, plants, and microorganisms.

The conventional bioassay tests conducted simultaneously with the Microtox™ screening test for the 20 stormwater sheetflow and combined sewer overflow (CSO) samples were all short-term tests. However, some of the tests were indicative of chronic toxicity (e.g., life cycle tests and the marine organism sexual reproduction tests), whereas the others would be classically considered as indicative of acute toxicity (e.g., Microtox™ and the fathead minnow tests). The following list shows the major tests that were conducted by each participating laboratory:

1. University of Alabama at Birmingham, Environmental Engineering Laboratory
   Microtox™ bacterial luminescence tests (10-, 20-, and 35-minute exposures) using the marine *Photobacterium phosphoreum.*

2. Wright State University, Biological Sciences Department

    Macrofaunal toxicity tests:

        Daphnia magna (water flea) survival; *Lemma minor* (duckweed) growth; and *Selenastrum capricornutum* (green alga) growth.

    Microbial activity tests (bacterial respiration):

        Indigenous microbial electron transport activity;
        Indigenous microbial inhibition of β-galactosidase activity;
        Alkaline phosphatase for indigenous microbial activity;
        Inhibition of β-galactosidase for indigenous microbial activity; and
        Bacterial surrogate assay using *O*-nitrophenol-β-D-galactopyranside activity and *Escherichia coli.*

3. EPA Environmental Research Laboratory, Duluth, MN

    Ceriodaphnia dubia (water flea) 48-h survival; and
    Pimephales promelas (fathead minnow) 96-h survival.

4. EPA Environmental Research Laboratory, Narragansett Bay, RI

    Champia parvula (marine red alga) sexual reproduction (formation of cystocarps after 5 to 7 d exposure); and
    Arbacua punctulata (sea urchin) fertilization by sperm cells.

Table 4-18 summarizes the results of the toxicity tests. The *C. dubia. P. promelas,* and C. *Parvula* tests experienced problems with the control samples and, therefore, these results are therefore uncertain. The *A. pustulata* tests on the stormwater samples also had a potential problem with the control samples. The CSO test results (excluding the fathead minnow tests) indicated that from 50% to 100% of the samples were toxic, with most tests identifying the same few samples as the most toxic. The toxicity tests for the stormwater samples indicated that 0% to 40% of the samples were toxic. The Microtox™ screening procedure gave similar rankings for the samples as the other toxicity tests.

Laboratory toxicity tests can result in important information on the effects of stormwater in receiving waters, but actual in-stream taxonomic studies should also be conducted. A recently published proceedings of a conference on stormwater impacts on receiving streams (Herricks 1995) contains many examples of actual receiving water impacts and toxicity test protocols for stormwater.

Table 4-18. Fraction of samples rated as toxic.

Sample series	Combined sewer overflows (%)	Stormwater (%)
Microtox™ marine bacteria	100	20
C. Dubia	60	0[1]
P. promelas	0[1]	0[1]
C. parvula	100	0[1]
A. punctulata	100	0[1]
D. magna	63	40
L. minor	50[1]	0

1) Results uncertain, see text

All of the Birmingham samples represented separate stormwater. However, as part of the Microtox™ evaluation, several CSO samples from New York City were also tested to compare the different toxicity tests. These samples were collected from six CSO discharge locations having the following land uses:

1. 290 acres, 90% residential and 10% institutional.
2. 50 acres, 100% commercial.
3. 620 acres, 20% institutional, 6% commercial, 5% warehousing, 5% heavy industrial, and 64% residential.
4. 225 acres, 13% institutional, 4% commercial, 2% heavy industrial. and 81% residential.
5. 400 acres, 1% institutional and 99% residential.
6. 250 acres, 88% commercial. 6% warehousing, and 6% residential.

Therefore, there was a chance that some of the CSO samples may have had some industrial process waters. However, none of the Birmingham sheetflow samples could have contained any process waters because of how and where they were collected.

The Microtox™ screening procedure gave similar toxicity rankings for the 20 samples as the conventional bioassay tests. It is also a rapid procedure (requiring about one hour) and only requires small (<1 ml) sample volumes. The Microtox™ toxicity test uses marine bioluminescence bacteria and monitors the light output for different sample concentrations. About one million bacteria organisms are used per sample, resulting in highly repeatable results. The more toxic samples produce greater stress on the bacteria test organisms that results in a greater light attenuation compared to the control sample. Note that the Microtox™ procedure was not used during this research to determine the absolute toxicities of the samples or to predict the toxic effects of stormwater runoff on receiving waters. It was used to compare the relative toxicities of different samples that may indicate efficient source area treatment locations, and to examine changes in toxicity during different treatment procedures.

Potential Sources

A drainage system captures runoff and pollutants from many source areas, all with individual characteristics influencing the quantity of runoff and pollutant load. Impervious source areas may contribute most of the runoff during small storm events (e.g., paved parking lots, streets, driveways, roofs, and sidewalks). Pervious source areas can have higher material washoff potentials and become important contributors for larger storm events when their infiltration rate capacity is exceeded (e.g., gardens, bare ground, unpaved parking areas, construction sites, undeveloped areas). Many other factors also affect the pollutant contributions from source areas, including: surface roughness, vegetative cover, gradient and hydraulic connections to a drainage system; rainfall intensity, duration, and antecedent dry period; and pollutant availability due to direct contamination from local activities, cleaning frequency/efficiency, and natural and regional sources of pollutants. The relative importance of the different source areas is therefore a function of the area characteristics, pollutant washoff potential, and the rainfall characteristics (Pitt 1987).

Important sources of toxicants are often related to the land use (e.g., high traffic capacity roads, industrial processes, and storage area) that are unique to specific land uses activities. Automobile related sources affect the quality and quantity of road dust particles through gasoline and oil drips/spills, deposition of exhaust products, and wear of tire, brake, and pavement materials (Shaheen 1975). Urban landscaping practices potentially produce vegetation cuttings and fertilizer and pesticide washoff. Miscellaneous sources include holiday firework debris, wildlife and domestic pet wastes, and possible sanitary wastewater infiltration. In addition, resuspension and deposition of pollutants/particles via the atmosphere can increase or decrease the contribution potential of a source area (Pitt and Bozeman 1982, Bannerman et al. 1993).

Results

Table 4-19 summarizes the source area sample data for the most frequently detected organic toxicants and for all of the metallic toxicants analyzed. The organic toxicants analyzed, but not reported, were generally detected in five, or less, of the non-filtered samples and in none of the filtered samples. Table 4-19 shows the mean, maximum, and minimum concentrations for the detected toxicants. Note that these values are based only on the observed concentrations. They do not consider the non-detectable conditions. Mean values based on total sample numbers for each source area category would therefore result in much lower concentrations. The frequency of detection is therefore an important consideration when evaluating organic toxicants. High detection frequencies for the organics may indicate greater potential problems than infrequent high concentrations.

Table 4-19 also summarizes the measured pH and SS concentrations. Most pH values were in the range of 7.0 to 8.5 with a low of 4.4 and a high of 11.6 for roof and concrete plant storage area runoff samples, respectively. This range of pH can have dramatic effects on the speciation of the metals analyzed. The SS concentrations were generally less than 100 mg/l, with impervious area runoff (e.g., roofs and parking areas) having much lower SS concentrations and turbidities compared to samples obtained from pervious areas (e.g., landscaped areas).

Out of more than 35 targeted compounds analyzed, 13 were detected in more than 10% of all samples, as shown in Table 4-19. The greatest detection frequencies were for 1,3-dichlorobenzene and fluoranthene, which were each detected in 23% of the samples. The organics most frequently found in these source area samples (i.e., polycyclic aromatic hydrocarbons (PAH), especially fluoranthene and pyrene) were similar to the organics most frequently detected at outfalls in prior studies (EPA 1983a).

Roof runoff, parking area and vehicle service area samples had the greatest detection frequencies for the organic toxicants. Vehicle service areas and urban creeks had several of the observed maximum organic compound concentrations. Most of the organics were associated with the non-filtered sample portions, indicating an association with the particulate sample fractions. The compound 1,3-dichlorobenzene was an exception, having a significant dissolved fraction.

In contrast to the organics, the heavy metals analyzed were detected in almost all samples, including the filtered sample portions. The non-filtered samples generally had much higher concentrations, with the exception of zinc, which was mostly associated with the dissolved sample portion (i.e., not associated with the SS). Roof runoff generally had the highest concentrations of zinc, probably from galvanized roof drainage components, as previously reported by Bannerman et al. (1983). Parking and storage areas had the highest nickel concentrations, while vehicle service areas and street runoff had the highest concentrations of cadmium and lead. Urban creek samples had the highest copper concentrations, which were probably due to illicit industrial connections or other non-stormwater discharges.

Table 4-20 shows the relative toxicities of the collected stormwaters. A wide range of toxicities was found. About 9% of the non-filtered samples were considered highly toxic using the Microtox™ toxicity screening procedure. About 32% of the samples were moderately toxic and about 59% were considered non-toxic. The greatest percentage of samples considered the most toxic were from industrial storage and parking areas. Landscaped areas also had a high incidence of highly toxic samples (presumably due to landscaping chemicals) and roof runoff had some highly toxic samples (presumably due to high zinc concentrations). Treatability study activities indicated that filtering the samples through a range of fine sieves and finally a 0.45µm filter consistently reduced sample toxicities. The chemical analyses also generally found much higher toxicant concentrations in the non-filtered sample portions, compared to the filtered sample portions.

Table 4-19. Stormwater toxicants detected in at least 10% of the source area sheetflow samples (µg/l, unless otherwise noted).

	Roof areas		Parking areas		Storage areas		Street runoff		Loading docks		Vehicle service areas		Landscaped areas		Urban creeks		Detention ponds	
	N.F.[1]	F.[2]	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.
Total samples	12	12	16	16	8	8	6	6	3	3	5	5	6	6	19	19	12	12
Base neutrals (detection limit = 0.5 µg/l)																		
1,3-Dichlorobenzene detection frequency = 20% N.F. and 13% F.																		
No. detected[3]		2	3	2	1	1	1	1	0	0	3	2	3	2	2	0	1	1
Mean[4]	52	20	34	13	16	14	5.4	3.3			48	26	29	5.6	93		27	21
Max.	88	23	103	26							72	47	54	7.5	120			
Min.[5]	14	17	3.0	2.0							6.0	4.9	4.5	3.8	65			
Fluoranthene detection frequency = 20% N.F. and 12% F.																		
No. detected	3	2	3	2	1	0	1	1	0	0	3	2	3	2	1	0	2	1
Mean	23	9.3	37	2.7	4.5	0	0.6	0.5			39	3.6	13	1.0	130		10	6.6
Max.	45	14	110	5.4							53	6.8	38	1.3			14	
Min.	7.6	4.8	3.0	2.0							0.4	0.4	0.7	0.7			6.6	
Pyrene detection frequency = 17% N,F, and 7% F.																		
No. detected	1	0	3	2	1	0	1	1	0	0	3	2	2	0	1	0	2	1
Mean	28		40	9.8	8		1.0	0.7			44	4.1	5.3		100		31	5.8
Max.			120	20							51	7.4	8.2				57	
Min.			3.0	2.0							0.7	0.7	2.3				6.0	
Benzo(b)fluoranthene detection frequency = 15% N.F. and 0% F.																		
No. detected	4	0	3	0	0	0	1	0	0	0	2	0	1	0	2	0	0	0
Mean	76		53				14				98		30		36			
Max.	260		160								110				64			
Min.	6.4		3.0								90				8.0			
Benzo(k)fluoranthene detection frequency = 11% N.F. and 0% F.																		
No. detected	0	0	3	0	0	0	1	0	0	0	2	0	1	0	2	0	0	0
Mean			20				15				59		61		55			
Max.			1								103				78			
Min.			3.0								15				31			
Benzo(a)pyrene detection frequency = 15% N.F. and 0% F.																		
No. detected	4	0	3	0	0	0	1	0	0	0	2	0	1	0	2	0	0	0
Mean	99		40				19				90		54		73			
Max.	300		120								120				130			
Min.	34		3.0								60				19			

Table 4-19. Stormwater toxicants detected in at least 10% of the source area sheetflow samples (μg/l, unless otherwise noted).Continued.

	Roof areas		Parking areas		Storage areas		Street runoff		Loading docks		Vehicle service areas		Landscaped areas		Urban creeks		Detention ponds	
	N.F.[1]	F.[2]	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.
Total samples	12	12	16	16	8	8	6	6	3	3	5	5	6	6	19	19	12	12
Bis(2-chloroethyl) ether detection frequency = 12% N.F. and 2% F.																		
No. detected		1	2	0	0	0	1	0	0	0	1	1	1	0	1	0	1	0
Mean	42	17	20				15				45	23	56		200		15	
Max.	87	2	39															
Min.	20		2.0								6.0	4.9	4.5	3.8	65			
Bis(chloroisopropyl) ether detection frequency = 13% N.F. and 0% F.																		
No. detected	3	0	3	0	0	0	0	0	0	0	2	0	1	0	2	0	0	0
Mean	99		130								120		85		59			
Max.	150		400								160				78			
Min.	68		3.0								74				40			
Naphthalene detection frequency = 11% N.F. and 6% F.																		
No. detected	2	0	1	1	0	0	0	0	0	0	2	1	1	0	1	1	2	2
Mean	17		72	6.6							70	82	49		300	6.7	43	12
Max.	21										100						68	17
Min.	13										37						18	6.6
Benzo(a)anthracene detection frequency = 10% N.F. and 0% F.																		
No. detected	1	0	3	0	0	0	0	0	0	0	2	0	1	0	1	0	0	0
Mean	16		24								35		54		61			
Max.			73								39							
Min.			3.0								31							
Butylbenzyl phthalate detection frequency = 10% N.F. and 4% F.																		
No. detected	1	0	2	1	0	0	0	0	0	0	2	2	1	0	1	0	1	0
Mean	100		12	3.3							26	9.8	130		59		13	
Max.			21								48	16						
Min.			3.3								3.8	3						
Pesticides (detection limit = 0.3 μg/l)																		
Chlordane detection frequency = 11% N.F. and 0% F.																		
No. detected	2	0	2	0	3	0	1	0	0	0	1	0	0	0	0	0	0	0
Mean	1.6		1.0		1.7		0.8			0.8								
Max.	2.2		1.2		2.9													
Min.	0.9		0.8		1.0													

Table 4-19. Stormwater toxicants detected in at least 10% of the source area sheetflow samples (µg/l, unless otherwise noted).Continued.

	Roof areas		Parking areas		Storage areas		Street runoff		Loading docks		Vehicle service areas		Landscaped areas		Urban creeks		Detention ponds	
	N.F.[1]	F.[2]	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.
Total samples	12	12	16	16	8	8	6	6	3	3	5	5	6	6	19	19	12	12
Metals (detection limit = 1 µg/l)																		
Lead detection frequency = 100% N.F. and 54% F.																		
No. detected		1	16	8	8	7	6	4	3	1	5	2	6	1	19	15	12	8
Mean	41	1.1	46	2.1	105	2.6	43	2.0	55	2.3	63	2.4	24	1.7	20	1.4	19	1.0
Max.	170		130	5.2	330	5.7	150	3.9	80		110	3.4	70		100	1.6	55	1.0
Min.	1.3		1.0	1.2	3.6	1.6	1.5	1.1	25		27	1.4	1.4		1.4	<1	1	<1
Zinc detection frequency = 99% N.F. and 98% F.																		
No. detected	12	12	16	16	8	7	6	6	2	2	5	5	6	6	19	19	12	12
Mean	250	220	110	86	1730	22	58	31	55	33	105	73	230	140	10	10	13	14
Max.	1580	1550	650	560	13100	100	130	76	79	62	230	230	1160	670	32	23	25	25
Min.	11	9	12	6	12	3.0	4.0	4.0	31	4.0	30	11	18	18	<1	<1	<1	<1
Copper detection frequency = 98% N.F. and 78% F.																		
No. detected	11	7	15	13	8	6	6	5	3	2	5	4	6	6	19	17	12	8
Mean	110	2.9	116	11	290	250	280	3.8	22	8.7	135	8.4	81	4.2	50	1.4	43	20
Max.	900	8.7	770	61	1830	1520	1250	11	30	15	580	24	300	8.8	440	1.7	210	35
Min.	1.5	1.1	10	1.1	10	1.0	10	1.0	15	2.6	1.5	1.1	1.9	0.9	<1	<1	0.2	<1
Aluminum detection frequency = 97% N.F. and 92% F.																		
No. detected	12	12	15	15	7	6	6	6	3	1	5	4	5	5	19	19	12	12
Mean	6850	230	3210	430	2320	180	3080	880	780	18	700	170	2310	1210	620	190	700	210
Max.	71300	1550	6480	2890	6990	740	10040	4380	930		1370	410	4610	1860	3250	500	1570	360
Min.	25	6.4	130	5.0	180	10	70	18	590		93	0.3	180	120	<5	<5	<5	<5
Cadmium detection frequency = 95% N.F. and 69% F.																		
No. detected	11	7	15	9	8	7	6	5	3	3	5	3	4	2	19	15	12	9
Mean	3.4	0.4	6.3	0.6	5.9	2.1	37	0.3	1.4	0.4	9.2	0.3	0.5	0.6	8.3	0.2	2	0.5
Max.	30	0.7	70	1.8	17	10	220	0.6	2.4	0.6	30	0.5	1	1	30	0.3	11	0.7
Min.	0.2	0.1	0.1	0.1	0.9	0.3	0.4	0.1	0.7	0.3	1.7	0.2	0.1	0.1	<0.1	<0.1	0.1	0.4
Chromium detection frequency = 91% N.F. and 55% F.																		
No. detected	7	2	15	8	8	5	5	4	3	0	5	1	6	5	19	15	11	8
Mean	85	1.8	56	2.3	75	11	9.9	1.8	17		74	2.5	79	2.0	62	1.6	37	2.0
Max.	510	2.3	310	5.0	340	32	30	2.7	40		320		250	4.1	710	4.3	230	3.0
Min.	5.0	1.4	2.4	1.1	3.7	1.1	2.8	1.3	2.4		2.4		2.2	1.4	<0.1	<0.1	<0.1	<0.1

Table 4-19. Stormwater toxicants detected in at least 10% of the source area sheetflow samples (µg/l, unless otherwise noted).Continued.

	Roof areas		Parking areas		Storage areas		Street runoff		Loading docks		Vehicle service areas		Landscaped areas		Urban creeks		Detention ponds	
	N.F.[1]	F.[2]	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.	N.F.	F.
Total samples	12	12	16	16	8	8	6	6	3	3	5	5	6	6	19	19	12	12
Nickel detection frequency = 90% N.F. and 37% F.																		
No. detected		0	14	4	8	1	5	0	3	1	5	1	4	1	18	16	11	8
Mean	16		45	5.1	55	87	17		6.7	1.3	42	31	53	2.1	29	2.3	24	3.0
Max.	70		130	13	170		70		8.1		70		130		74	3.6	70	6.0
Min.	2.6		4.2	1.6	1.9		1.2		4.2		7.9		21		<1	<1	1.5	<1
Other constituents (always detected, analyzed only for non–filtered samples)																		
pH																		
Mean	6.9		7.3		8.5		7.6		7.8		7.2		6.7		7.7		8.0	
Max.	8.4		8.7		12		8.4		8.3		8.1		7.2		8.6		9.0	
Min.	4.4		5.6		6.5		6.9		7.1		5.3		6.2		6.9		7.0	
Suspended solids																		
Mean	14		110		100		49		40		24		33		26		17	
Max.	92		750		450		110		47		38		81		140		60	
Min.	0.5		9.0		5.0		7.0		34		17		8.0		5.0		3.0	

1) N.F.: concentration associated with a nonfiltered sample.
2) F.: concentration after the sample was filtered through a 0.45 µm membrane filter.
3) Number detected refers to the number of samples in which the toxicant was detected.
4) Mean values based only on the number of samples with a definite concentration of toxicant reported (not on the total number of samples analyzed).
5) The minimum values shown are the lowest concentration detected, they are not necessarily the detection limit.

Replicate samples were collected from several source areas at three land uses during four different storm events to statistically examine toxicity and pollutant concentration differences due to storm and site conditions. These data indicated that variations in Microtox™ toxicities and organic toxicant concentrations may be partially explained by rain characteristics. As an example, high concentrations of many of the PAHs were associated with long antecedent dry periods and large rains (Barron 1990).

Table 4-20. Relative toxicity of samples using Microtox™ (non-filtered).

Local Source Areas	Highly Toxic (%)	Moderately Toxic (%)	Not Toxic (%)	Number of Samples
Roofs	8	58	33	12
Parking Areas	19	31	50	16
Storage Areas	25	50	25	8
Streets	0	67	33	6
Loading Docks	0	67	33	3
Vehicle Service Areas	0	40	60	5
Landscaped Areas	17	17	66	6
Urban Creeks	0	11	89	19
Detention Ponds	8	8	84	12
All Areas	9	32	59	87

Microbics suggested toxicity definitions for 35 minute exposures:
- Highly toxic - light decrease >60%
- Moderately toxic - light decrease <60% & >20%
- Not toxic - light decrease <20%

References

Alley, W. M. (1980). Determination of the decay coefficient in the exponential washoff equation. International Symposium on Urban Runoff. University of Kentucky. Lexington, KY. July.

Alley, W. M. (1981). Estimation of impervious-area washoff parameters. Water Resources Research. Vol. 17, No. 4, pp 1161-1166.

American Public Works Association (1969). Water Pollution Aspects of Urban Runoff. Water Pollution Control Research Series WP-20-15. Federal Water Pollution Control Administration. January.

Bannerman, R., K. M. Baun, P. E. Bohn, and D. A. Graczyk (1983). Evaluation of Urban Nonpoint Source Pollution Management in Milwaukee County, Wisconsin. PB 84-114164. U.S. Environmental Protection Agency. Chicago, IL.

Bannerman, R., D. W. Owens, R. B. Dodds, and N. J. Hornewer (1993). Sources of pollutants in Wisconsin stormwater. Water Science and Technology. Vol. 28, No. 3-5, pp. 241-259.

Barkdoll, M. P., D. E. Overton, and R. P. Beton (1977). Some effects of dustfall on urban stormwater quality. Water Pollution Control Federation. 49(9):1976-84.

Barron, P. (1990). Characterization of Polynuclear Aromatic Hydrocarbons in Urban Runoff. Master's Thesis. The University of Alabama at Birmingham Department of Civil Engineering. Birmingham, AL.

Betson, R. P. (1978). Precipitation and streamflow quality relationships in an urban area. Water Resources Research. 14(6):1165-1169.

Box, G. E. P., W. G. Hunter, and J. S. Hunter (1978). Statistics for Experimenters. John Wiley and Sons. New York, NY.

COE (U.S. Corps of Engineers). Hydrologic Engineering Center. (1975). Urban Storm Water Runoff: STORM. Generalized Computer Program. 723-58-L2520. Davis, CA. May.

Cowherd, C. J., C. M. Maxwell, and D. W. Nelson (1977). Quantification of Dust Entrainment from Paved Roadways. EPA-450 3-77-027. U.S. Environmental Protection Agency. Research Triangle Park, NC. July.

Denver Regional Council of Governments (1983). Urban Runoff Quality in the Denver (Colorado) Region. Prepared for the U.S. EPA. Washington, DC. PB85-101640. September.

Donigian, A. S., Jr. and N.H. Crawford (1976). Modeling Nonpoint Pollution from the Land Surface. EPA-600/3-76-083. U.S. Environmental Protection Agency. Athens, GA. July.

Durum, W. H. (1974). Occurrence of some trace metals in surface waters and groundwaters. In Proceeding of the Sixteenth Water Quality Conference. Am. Water Works Assoc., et al. Univ. of Illinois Bull. 71(108). Urbana, IL.

EPA (1982). Methods for Organic Chemical Analyses of Municipal and Industrial Wastewater. Environmental Monitoring and Data Support Laboratory. EPA-600/4-82-057. U.S. Environmental Protection Agency. Cincinnati, OH.

EPA (1983a). Results of the Nationwide Urban Runoff Program. Water Planning Division. PB 84-185552. Washington, D.C. December.

EPA (1983b). Methods for Chemical Analysis of Water and Wastes. EPA-600/4-79-020. U.S. Environmental Protection Agency. Cincinnati, OH.

Field, R., E.J. Struzeski, Jr., H.E. Masters and A.N. Tafuri (1973). Water Pollution and Associated Effects from Street Salting. EPA-R2-73-257. U.S. Environmental Protection Agency. Cincinnati, OH. May.

Gupta, M., D. Mason, M. Clark, T. Meinholz, C. Hansen, and A Geinopolos (1977). Screening Flotation Treatment of Combined Sewer Overflows Volume I - Bench Scale and Pilot Plant Investigations. EPA-600/2-77-069a. U.S. Environmental Protection Agency. Cincinnati, OH. August.

Herricks, E. E. (1995). Stormwater Runoff and Receiving Systems: Impact, Monitoring, and Assessment. CRC/Lewis Publishers. Boca Raton, FL.

Huber, W.C. and J.P. Heaney. (1981). The USEPA Storm Water Management Model, SWMM: A ten-year Perspective. Second international Conference on Urban Storm Drainage. Urbana, IL. June.

Jewell, T.K., D.D. Adrian and D.W. Hosmer (1980). Analysis of stormwater pollutant washoff estimation techniques. International Symposium on Urban Storm Runoff. University of Kentucky. Lexington, KY. July.

Kobriger, N.P., T.L. Meinholiz, M.K. Gupta, and R.W. Agnew (1981). Constituents of Highway Runoff. Vol. 3. Predictive Procedure for Determining Pollution Characteristics in Highway Runoff. FHWA/RD-81/044. Federal Highway Administration. Washington, D.C. February.

Koeppe, D. E. (1977). comp. Vol. IV: Soil-water-air-plant studies. In: Environmental Contamination by Lead and Other Heavy Metals. G. L. Rolfe and K. A. Peinbold, eds. Institute for Environmental Studies. Univ. of Illinois. Urbana-Champaign, IL. July.

Lindsay, W. L. (1979). Chemical Equilibria in Soils. John Wiley and Sons. New York, NY.

Malmquist, Per-Arne (1978). Atmospheric Fallout and Street Cleaning - Effects on Urban Stream Water and Snow. Prog. Wat Tech., 10(5/6): 495-505. Pergamon Press. Great Britain. September.

Manning, M.J., R.H. Sullivan, and T.M. Kipp (1976). Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges. Vol. III: Characterization of Discharges. U.S. Environmental Protection Agency. Cincinnati, OH. October.

Murphy, W. (1975). Roadway Particulate Losses: American Public Works Assoc. Unpublished.

Novotny, V. and G. Chesters (1981). Handbook of Nonpoint Pollution Sources and Management. Van Norstrand Reinhold Company. New York, NY.

PEDCo-Environmental, Inc. (1977). Control of Re-entrained Dust from Paved Streets. EPA-907/9-77-007. U.S. Environmental Protection Agency. Kansas City, MO.

Phillips, G. R., and R. C. Russo (1978). Metal Bioaccumulation in Fishes and Aquatic Invertebrates: A Literature Review. EPA-600-3-78-103, U.S. Environmental Protection Agency. Duluth, MN. December.

Pitt, R. (1979). Demonstration of Nonpoint Pollution Abatement Through Improved Street Cleaning Practices. EPA-600/2-79-161. U.S. Environmental Protection Agency. Cincinnati, OH. August.

Pitt, R. (1983). Urban Bacteria Sources and Control in the Lower Rideau River Watershed. Ottawa, Ontario. Ontario Ministry of the Environment. ISBN 0-7743-8487-5. 165 pgs.

Pitt, R. (1985). Characterizing and Controlling Urban Runoff through Street and Sewerage Cleaning. U.S. Environmental Protection Agency. Storm and Combined Sewer Program. Risk Reduction Engineering Laboratory. EPA/600/S2-85/038. PB 85-186500. Cincinnati, OH. June.

Pitt, R. (1987). Small Storm Urban Flow and Particulate Washoff Contributions to Outfall Discharges. Ph.D. dissertation submitted to the Department of Civil and Environmental Engineering. University of Wisconsin - Madison.

Pitt R. and M. Bozeman (1982). Sources of Urban Runoff Pollution and Its Effects on an Urban Creek. EPA 600/S2-82-090. U.S. Environmental Protection Agency. Cincinnati, OH.

Pitt, R. and G. Shawley (1982). A Demonstration of Non-Point Source Pollution Management on Castro Valley Creek. Alameda County Flood Control and Water Conservation District (Hayward, CA) for the Nationwide Urban Runoff Program. U.S. Environmental Protection Agency. Water Planning Division. Washington, D.C. June.

Pitt, R. and R. Sutherland (1982). Washoe County Urban Stormwater Management Program. Volume 2, Street Particulate Data Collection and Analyses. Washoe Council of Governments. Reno, NV. August.

Pin, R. and J. McLean (1986). Toronto Area Watershed Management Strategy Study. Humber River Pilot Watershed Project. Ontario Ministry of the Environment. Toronto, Ontario.

Pitt, R., M. Lalor, R. Field, D.D. Adrian, and D. Barbe (1993). Investigation of Inappropriate Pollutant Entries into Storm Drainage Systems, A User's Guide. EPA/600/R-92/238. U.S. Environmental Protection Agency. Cincinnati, OH.

Rolfe, G.L. and K.A. Reinhold (1977). Vol. I.- Introduction and Summary. Environmental Contamination by Lead and Other Heavy Metals. Institute for Environmental Studies. University of Illinois. Champaign-Urbana, IL. July.

Rubin, A. J., ed. (1976). Aqueous-Environmental Chemistry of Metals. Ann Arbor Science Publishers. Ann Arbor, MI.

Sartor J. and G. Boyd (1972). Water Pollution Aspects of Street Surface Contaminants. EPA-R2-72-081, U.S. Environmental Protection Agency. November.

Shaheen, D.G. (1975). Contributions of Urban Roadway Usage to Water Pollution. 600/2-75-004. U.S. Environmental Protection Agency. Washington, D.C. April.

Shelley, P.E. and D.R. Gaboury (1986). Estimation of pollution from highway runoff - initial results. Conference on Urban Runoff Quality - Impact and Quality Enhancement Technology. Henniker, NH. Edited by B. Urbonas and L.A. Roesner. Proceedings published by the American Society of Civil Engineering. New York, NY. June.

Shen, H.W. (1981). Some basic concepts on sediment transport in urban storm drainage systems. Second International Conference on Urban Storm Drainage. Urbana, IL. June.

Singer, M.J. and J. Blackard (1978). Effect of mulching on sediment in runoff from simulated rainfall. Soil Sci. Soc. Am. J., 42:481-486.

Solomon, R.L., and D.F.S. Natusch (1977). Vol:lll: Distribution and characterization of urban dists. In: Environmental Contamination by Lead and Other Heavy Metals. G. L. Rolfe and K. G. Reinbold, eds. Institute for Environmental Studies. Univ. of Illinois. Urbana-Champaign, IL. July.

Spring, R. J., R. B. Howell, and E. Shirley (1978). Dustfall Analysis for the Pavement Storm Runoff Study (I-405 Los Angeles). Office of Transportation Laboratory. California Dept. of Transportation. Sacramento, CA. April.

Sutherland, R., and R.H. McCuen (1978). Simulation of urban nonpoint source pollution. Water Resources Bulletin. Vol. 14, No. 2, pp 409-428. April.

Sutherland, R., W. Alley, and F. Ellis (undated). Draft Users' Guide for Particulate Transport Model (PTM). CH2M –HILL. Portland, OR for the U.S. Geological Survey.

Terstriep, M.L., G.M. Bender, and D.C. Noel (1982). Final Report - NURP Project, Champaign, Illinois: Evaluation of the Effectiveness of Municipal Street Sweeping in the Control of Urban Storm Runoff Pollution. State Water Survey Division. Illinois Dept. of Energy and Natural Resources. Champaign-Urbana, IL. December.

Verschueren, K. (1983). Handbook of Environmental Data on Organic Chemicals, 2nd edition. Van N Reinhold Co. New York, NY.

Wilber, W. G., and J.V. Hunter (1980). The Influence of Urbanization on the Transport of Heavy Metals in New Jersey Streams. Water Resources Research Institute. Rutgers University. New Brunswick, NJ.

Yalin, M.S. (1963). An expression for bed load transportation. Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers. Vol 89, pp 221-250.

Chapter 5

Receiving Water and Other Impacts

Robert Pitt

Desired Water Uses Versus Stormwater Impacts

The main purpose of treating stormwater is to reduce its adverse impacts on receiving water beneficial uses. Therefore, this report on wet-weather flow management systems includes an assessment of the detrimental effects that runoff is actually having on a receiving water.

Urban receiving waters may have many beneficial use goals, including:

1. Stormwater conveyance (flood prevention).
2. Biological uses (e.g., warm water fishery, biological integrity).
3. Non-contact recreation (e.g., linear parks, aesthetics, boating).
4. Contact recreation (swimming).
5. Water supply and irrigation.

With full development in an urban watershed and with no stormwater controls, it is unlikely that any of these uses can be obtained. With less development and with the application of stormwater controls, some uses may be possible. Unreasonable expectations should not be placed on urban waters, because the cost to obtain these uses may be prohibitive. With full-scale development and lack of adequate stormwater controls, severely degraded streams will be common.

Stormwater conveyance and aesthetics should be the basic beneficial use goals for all urban waters. Biological integrity should also be a goal, but with the realization that the natural stream ecosystem will be severely modified with urbanization. Certain basic controls, installed at the time of development, plus protection of stream habitat, may enable partial realization of some of these basic goals in urbanized watersheds. Careful planning and optimal utilization of stormwater controls are necessary to obtain these basic goals in most watersheds. Water contact recreation, consumptive fisheries, and water supplies are not appropriate goals for most urbanized watersheds. These higher uses may be possible in urban areas where the receiving waters are large and drain mostly undeveloped areas.

In general, monitoring of urban stormwater runoff has indicated that the biological beneficial uses of urban receiving waters are most likely affected by habitat destruction and long-term pollutant exposures (especially to macroinvertebrates via contaminated sediment). Documented effects associated from acute exposures of toxicants in the water column are rare (Field and Pitt 1990, Pitt 1995).

Receiving water pollutant concentrations resulting from runoff events and typical laboratory bioassay test results have not indicated many significant short-term receiving water problems. As an example, Lee and Jones-Lee (1993) state that exceedences of numeric criteria by short-term discharges do not necessarily imply that a beneficial use impairment exists. Many toxicologists and water quality experts have concluded that the relatively short periods of exposures to the toxicant concentrations in stormwater are not sufficient to produce the receiving water effects that are evident in urban receiving waters, especially considering the relatively large portion of the toxicants that are associated with particulates (Lee and Jones-Lee 1995a and 1995b). Lee and Jones-Lee (1995a and 1995b) conclude that the biological problems evident in urban receiving waters due to stormwater discharges are mostly associated with illegal discharges and that the sediment bound toxicants are of little risk. Mancini and Plummer (1986) have long been advocates of numeric water quality standards for stormwater that reflect the partitioning of the toxicants and the short periods of exposure during rains. Unfortunately, this approach attempts to isolate individual runoff events and does not consider the accumulative adverse effects caused by the frequent exposures of receiving water organisms to stormwater (Davies 1995, Herricks 1995 and Herricks et al. 1996). Recent investigations have identified acute toxicity problems associated with short-term (about 10 to 20 day) exposures to adverse toxicant concentrations in urban receiving streams (Crunkilton et al. 1997). However, the most severe receiving water problems are likely associated with chronic exposures to contaminated sediment and to habitat destruction.

The effects of stormwater on receiving waters are very site specific. Accordingly, site investigations of local waters are highly recommended to understand the magnitude and like cause of the problems. Burton and Pitt (1996) have prepared a book that details site investigation procedures that can be used for local waters. The following is a summary of recent work describing the toxicological and ecological effects of stormwater.

Toxicological Effects of Stormwater

The need for endpoints for toxicological assessments using multiple stressors was discussed by Marcy and Gerritsen (1996). They used five watershed-level ecological risk assessments to develop appropriate endpoints based on specific project objectives. Dyer and White (1996) also examined the problem of multiple stressors affecting toxicity assessments. They felt that field surveys rarely can be used to verify simple single parameter laboratory experiments. They developed a watershed approach integrating numerous databases in conjunction with in-situ biological observations to help examine the effects of many possible causative factors. Toxic effect endpoints are additive for compounds having the same "mode of toxic action", enabling predictions of complex chemical mixtures in water, as reported by Environmental Science & Technology (1996a). According to EPA researchers at the Environmental Research Laboratory in Duluth, MN, there are about five or six major action groups that contain almost all of the compounds of interest in the aquatic environment. Much work still needs to be done, but these new developing tools may enable improved prediction of in-stream toxic effects of stormwater.

Ireland et al. (1996) found that exposure to ultraviolet (UV) radiation (natural sunlight) increased the toxicity of PAH contaminated urban sediments to C. dubia. The toxicity was removed when the UV wavelengths did not penetrate the water column to the exposed organisms. Toxicity was also reduced significantly in the presence of UV when the organic fraction of the stormwater was removed. Photo-induced toxicity occurred frequently during low flow conditions and wet weather runoff and was reduced during turbid conditions.

Johnson et al. (1996) and Herricks et al. (1996) describe a structured tier testing protocol to assess both short-term and long-term wet weather discharge toxicity that they developed and tested. The protocol recognizes that the test systems must be appropriate to the time-scale of exposure during the discharge. Therefore, three time-scale protocols were developed, for intra-event, event, and long-term exposures. The use of standard whole effluent toxicity (WET) tests were found to over-estimate the potential toxicity of stormwater discharges.

The effects of stormwater on Lincoln Creek, near Milwaukee, WI, were described by Crunkilton et al. (1997). Lincoln Creek drains a heavily urbanized watershed of 19 mi^2 that is about nine miles long. On-site toxicity testing was conducted with side-stream flow-through aquaria using fathead minnows, plus in-stream biological assessments, along with water and sediment chemical measurements. In the basic tests, Lincoln Creek water was continuously pumped through the test tanks, reflecting the natural changes in water quality during both dry and wet weather conditions. The continuous flow-through mortality tests indicated no toxicity until after about 14 days of exposure, with more than 80% mortality after about 25 days, indicating that short-term toxicity tests likely underestimate stormwater toxicity. The biological and physical habitat assessments supported a definitive relationship between degraded stream ecology and urban runoff.

Rainbow (1996) presented a detailed overview of heavy metals in aquatic invertebrates. He concluded that the presence of a metal in an organism couldn't tell us directly whether that metal is poisoning the organism. However, if compared to concentrations in a suite of well-researched biomonitors, it is possible to determine if the accumulated concentrations are atypically high, with a possibility that toxic effects may be present. Allen (1996) also presented an overview of metal contaminated aquatic sediments. Allen's book presents many topics that would enable the user to better interpret measured heavy metal concentrations in urban stream sediments.

Ecological Effects of Stormwater

A number of comprehensive and long-term studies of biological beneficial uses in areas not affected by conventional point source discharges have typically shown impairments caused by urban runoff. The following paragraphs briefly describe a variety of such studies.

Klein (1979) studied 27 small watersheds having similar physical characteristics, but having varying land uses, in the Piedmont region of Maryland. During an initial phase of the study, they found definite relationships between water quality and land use. Subsequent study phases examined aquatic life relationships in the watersheds. The principal finding was that stream aquatic life problems were first identified with watersheds having imperviousness areas comprising at least 12 percent of the watershed. Severe problems were noted after the imperviousness quantities reached 30 percent.

Receiving water impact studies were also conducted in North Carolina (Lenet et al. 1979, Lenet and Eagleson 1981, Lenet et al. 1981). The benthic fauna occurred mainly on rocks. As sedimentation increased, the amount of exposed rocks decreased, with a decreasing density of benthic macroinvertebrates. Data from 1978 and 1979 in five cities showed that urban streams were grossly polluted by a combination of toxicants and sediment. Chemical analyses, without biological analyses, would have underestimated the severity of the problems because the water column quality varied rapidly, while the major problems were associated with sediment quality and effects on macroinvertebrates. Macroinvertebrate diversities were severely reduced in the urban streams, compared to the control streams. The biotic indices indicated very poor conditions for all urban streams. Occasionally, high populations of pollutant tolerant organisms were found in the urban streams, but would abruptly disappear before subsequent sampling efforts. This was probably caused by intermittent discharges of spills or illegal dumpings of toxicants. Although the cities studied were located in different geographic areas of North Carolina, the results were remarkably uniform.

During the Coyote Creek, San Jose, CA, receiving water study, 41 stations were sampled in both urban and nonurban perennial flow stretches of the creek over three years. Short and long-term sampling techniques were used to evaluate the effects of urban runoff on water quality, sediment properties, fish, macroinvertebrates, attached algae, and rooted aquatic vegetation (Pitt and Bozeman 1982). These investigations found distinct differences in the taxonomic composition and relative abundance of the aquatic biota present. The non-urban sections of the creek supported a comparatively diverse assemblage of aquatic organisms including an abundance of native fishes and numerous benthic macroinvertebrate taxa. In contrast, however, the urban portions of the creek (less than 5% urbanized) affected only by urban runoff discharges and not industrial or municipal discharges, had an aquatic community generally lacking in diversity and was dominated by pollution-tolerant organisms such as mosquitofish and tubificid worms.

A major nonpoint runoff receiving water impact research program was conducted in Georgia (Cook et al. 1983). Several groups of researchers examined streams in major areas of the state. Benke et al. (1981) studied 21 stream ecosystems near Atlanta having watersheds of one to three square miles each and land uses ranging from 0 to 98% urbanization. They measured stream water quality but found little relationship between water quality and degree of urbanization. The water quality parameters also did not identify a major degree of pollution. In contrast, there were major correlations between urbanization and the number of species found. They had problems applying diversity indices to their study because the individual organisms varied greatly in size (biomass).

CTA (1983) also examined receiving water aquatic biota impacts associated with urban runoff sources in Georgia. They studied habitat composition, water quality, macroinvertebrates, periphyton, fish, and toxicant concentrations in the water, sediment, and fish. They found that the impacts of land use were the greatest in the urban basins. Beneficial uses were impaired or denied in all three urban basins studied. Fish were absent in two of the basins and severely restricted in the third. The native macroinvertebrates were replaced with pollution tolerant organisms. The periphyton in the urban streams were very different from those found in the control streams and were dominated by species known to create taste and odor problems.

Pratt et al. (1981) used basket artificial substrates to compare benthic population trends along urban and nonurban areas of the Green River in Massachusetts. The benthic community became increasing disrupted as urbanization increased. The problems were not only associated with times of heavy rain, but seemed to be affected at all times. The stress was greatest during summer low flow periods and was probably localized near the stream bed. They concluded that the high degree of correspondence between the known sources of urban runoff and the observed effects on the benthic community was a forceful argument that urban runoff was the causal agent of the disruption observed.

Cedar swamps in the New Jersey Pine Barrens were studied by Ehrenfeld and Schneider (1983). They examined nineteen wetlands subjected to varying amounts of urbanization. Typical plant species were lost and replaced by weeds and exotic plants in urban runoff affected wetlands. Increased uptakes of phosphorus and lead in the plants were found. The researchers concluded that the presence of stormwater runoff to the cedar swamps caused marked changes in community structure, vegetation dynamics, and plant tissue element concentrations.

Medeiros and Coler (1982) and Medeiros et al. (1984) used a combination of laboratory and field studies to investigate the effects of urban runoff on fathead minnows. Hatchability, survival, and growth were assessed in the laboratory in flow-through and static bioassay tests. Growth was reduced to one half of the control growth rates at 60% dilutions of urban runoff. The observed effects were believed to be associated with a combination of toxicants.

The University of Washington (Pederson 1981, Richey et al. 1981, Perkins 1982, Richey 1982, Scott et al. 1982, Ebbert et al. 1983, Pitt and Bissonnette 1983, and Prych and Ebbert undated) conducted a series of studies to contrast the biological and chemical conditions in urban Kelsey Creek with rural Bear Creek in Bellevue, WA. The urban creek was significantly degraded when compared to the rural creek, but still supported a productive, but limited and unhealthy salmonid fishery. Many of the fish in the urban creek, however, had respiratory anomalies. The urban creek was not grossly polluted, but flooding from urban developments had increased dramatically in recent years. These increased flows markedly changed the urban stream's channel by causing unstable conditions with increased stream bed movement, and by altering the availability of food for the aquatic organisms. The aquatic organisms were very dependent on the few relatively undisturbed reaches. Dissolved oxygen concentrations in the sediments depressed embryo salmon survival in the urban creek. Various organic and metallic priority pollutants were discharged to the urban creek, but most of them were apparently carried through the creek system by the high storm flows to Lake Washington. The urbanized Kelsey Creek also had higher water temperatures (probably due to reduced shading) than Bear Creek. This probably caused the faster fish growth in Kelsey Creek.

The fish population in the urbanized Kelsey Creek had adapted to its degrading environment by shifting the species composition from coho salmon to less sensitive cutthroat trout and by making extensive use of less disturbed refuge areas. Studies of damaged gills found that up to three-fourths of the fish in Kelsey Creek were affected with respiratory anomalies, while no cutthroat trout and only two of the coho salmon sampled in the forested Bear Creek had damaged gills. Massive fish kills in Kelsey Creek and its tributaries were also observed on several occasions during the project due to the dumping of toxic materials down the storm drains.

There were also significant differences in the numbers and types of benthic organisms found in urban and forested creeks during the Bellevue research. Mayflies, stoneflies, caddisflies, and beetles were rarely observed in the urban Kelsey Creek, but were quite abundant in the forested Bear Creek. These organisms are commonly regarded as sensitive indicators of environmental degradation. One example of degraded conditions in Kelsey Creek was shown by a specie of clams (Unionidae) that was not found in Kelsey Creek, but was commonly found in Bear Creek. These clams are very sensitive to heavy siltation and unstable sediments. Empty clam shells, however, were found buried in the Kelsey Creek sediments indicating their previous presence in the creek and their inability to adjust to the changing conditions. The benthic organism composition in Kelsey Creek varied radically with time and place while the organisms were much more stable in Bear Creek.

Urban runoff impact studies were conducted in the Hillsborough River near Tampa Bay, FL, as part of the U.S. EPA’s Nationwide Urban Runoff Program (NURP) (Mote Marine Laboratory 1984). Plants, animals, sediment, and water quality were all studied in the field and supplemented by laboratory bioassay tests. Effects of salt water intrusion and urban runoff were both measured because of the estuarine environment. During wet weather, freshwater species were found closer to Tampa Bay than during dry weather.

In coastal areas, these additional natural factors made it even more difficult to identify the cause and effect relationships for aquatic life problems. During another NURP project, Striegl (1985) found that the effects of accumulated pollutants in Lake Ellyn (Glen Ellyn, IL) inhibited desirable benthic invertebrates and fish and increased undesirable phyotoplankton blooms.

The number of benthic organism taxa in Shabakunk Creek in Mercer County, NJ, declined from 13 in relatively undeveloped areas to four below heavily urbanized areas (Garie and McIntosh 1986 and 1990). Periphyton samples were also analyzed for heavy metals with significantly higher metal concentrations found below the heavily urbanized area than above.

Many of the above noted biological effects associated with urban runoff are likely caused by polluted sediments and benthic organism impacts. Examples of heavy metal and nutrient accumulations in sediments are numerous. In addition to the studies noted above, DePinto et al. (1980) found that the cadmium content of river sediments can be more than 1,000 times greater than the overlying water concentrations and the accumulation factors in sediments are closely correlated with sediment organic content. Another comprehensive study on polluted sediment was conducted by Wilber and Hunter (1980) along the Saddle River in New Jersey where they found significant increases in sediment contamination with increasing urbanization.

The effects of urban runoff on receiving water aquatic organisms or other beneficial uses is very site specific. Different land development practices create substantially different runoff flow characteristics. Different rain patterns cause different particulate washoff, transport and dilution conditions. Local attitudes also define specific beneficial uses and, therefore, current problems. There are also a wide variety of water types receiving urban runoff and these waters all have watersheds that are urbanized to various degrees. Therefore, it is not surprising that urban runoff effects, though generally dramatic, are also quite variable and site specific.

Claytor (1996a) summarized the approach developed by the Center for Watershed Protection as part of their EPA sponsored research on stormwater indicators (Claytor and Brown 1996). The 26 stormwater indicators used for assessing receiving water conditions were divided into six broad categories: water quality, physical/hydrological, biological, social, programmatic, and site. These were presented as tools to measure stress (impacting receiving waters), to assess the resource itself, and to indicate stormwater control program implementation effectiveness. The biological communities in Delaware’s Piedmont streams have been severely impacted by stormwater, after the extent of imperviousness in the watersheds exceeds about 8 to 15%, according to a review article by Claytor (1996b). If just conventional water quality measures are used, almost all (87%) of the state’s non-tidal streams supported their designated biological uses. However, when biological assessments are included, only 13% of the streams were satisfactory.

Changes in physical stream channel characteristics can have a significant effect on the biological health of the stream. Schueler (1996) stated that channel geometry stability can be a good indicator of the effectiveness of stormwater control practices. He also found that once a watershed area has more than about 10 to 15% effective impervious cover, noticeable changes in channel morphology occur, along with quantifiable impacts on water quality and biological conditions.

Stephenson (1996) studied changes in streamflow volumes in South Africa during urbanization. He found increased stormwater runoff, decreases in the groundwater table, and dramatically decreased times of concentration. The peak flow rates increased by about two-fold, about half caused by increased pavement (in an area having only about 5% effective impervious cover), with the remainder caused by decreased times of concentration.

Fate of Stormwater Pollutants in Surface Waters

Many processes may affect urban runoff pollutants after discharge. Sedimentation in the receiving water is the most common fate mechanism because many of the pollutants investigated are mostly associated with settleable particulate matter and have relatively low filterable concentration components. Exceptions include zinc and 1,3-dichlorobenzene, which are mostly associated with the filtered sample portions.

Particulate reduction can occur in many stormwater runoff and CSO control facilities, including (but not limited to) catchbasins, swirl concentrators, fine mesh screens, sand or other filters, drainage systems, and detention ponds. These control facilities (with the possible exception of drainage systems) allow reduction of the accumulated polluted sediment for final disposal in an appropriate manner. Uncontrolled sedimentation will occur in relatively quiescent receiving waters, such as lakes, reservoirs, or slow moving rivers or streams. In these cases, the wide dispersal of the contaminated sediment is difficult to remove and can cause significant detrimental effects on biological processes.

Biological or chemical degradation of the sediment toxicants may occur in the typically anaerobic environment of the sediment, but the degradation is quite slow for many of the pollutants. Degradation by photochemical reaction and volatilization (evaporation) of the soluble pollutants may also occur, especially when these pollutants are near the surface of aerated waters (Callahan et al. 1979, Parmer 1993). Increased turbulence and aeration encourages these degradation processes, which in turn may significantly reduce toxicant concentrations. In contrast, quiescent waters would encourage sedimentation that would also reduce water column toxicant concentrations, but increase sediment toxicant concentrations. Metal precipitation and sorption of pollutants onto suspended solids increases the sedimentation and/or floatation potential of the pollutants and also encourages more efficient bonding of the pollutants to soil particles, preventing their leaching to surrounding waters.

Receiving waters have a natural capacity to treat and/or assimilate polluted discharges. This capacity will be exceeded sooner (assuming equal inputs), resulting in more degradation, in smaller urban creeks and streams, than in larger receiving waters. Larger receiving waters may still have ecosystem problems from the long-term build up of toxicants in the sediment and repeated exposures to high flowrates, but these problems will be harder to identify using chemical analyses of the water alone, because of increased dilution (Pitt and Bissonnette 1983).

In-stream receiving water investigations of urban runoff effects need a multi-tiered monitoring approach, including habitat evaluations, water and sediment quality monitoring, flow monitoring, and biological investigations, conducted over long periods of time (Pitt 1991). In-stream taxonomic (biological community structure) investigations are needed to help identify actual toxicity problems. Laboratory bioassay tests can be useful to determine the major sources of toxicants and to investigate toxicity reduction through treatment, but they are not a substitute for actual in-stream investigations of receiving water effects. In order to identify the sources and treatability of the problem pollutants, detailed watershed investigations are needed, including both dry and wet weather urban drainage monitoring and source area monitoring.

An estimate of the actual pollutant loads (calculated from the runoff volumes and pollutant concentrations) from different watershed areas is needed for the selection and design of most treatment devices. Several characteristics of a source area are significant influences on the pollutant concentrations and stormwater runoff volumes. The washoff of debris, soil, and pollutants depends on the intensity of the rain, the properties of the material removed, and the surface characteristics where the material resides. The potential mass of pollutants available to be washed off will be directly related to the time interval between runoff events during which the pollutants can accumulate.

Human Health Effects of Stormwater

Water Environment & Technology (1996b) reported on an epidemiology study conducted at Santa Monica Bay, CA, that found that swimmers who swam in front of stormwater outfalls were 50% more likely to develop a variety of symptoms than those who swam 400 m from the same outfalls (Haile et al. 1996). This was a follow-up study after previous investigations found that human fecal waste was present in the stormwater collection systems. Environmental Science & Technology (1996b) also reported on this Santa Monica Bay study. They reported that more than 1% of the swimmers who swam in front of the outfalls were affected by fevers, chills, ear discharges, vomiting and coughing, based on surveys of more than 15,000 swimmers. The health effects were also more common for swimmers who were exposed on days when viruses were found in the outfall water samples.

Water Environment & Technology (1996a) reported that the fecal coliform counts decreased from about 500 counts/100 ml to about 150 counts/100 ml in the Mississippi River after the sewer separation program in the Minneapolis and St. Paul area of Minnesota. Combined sewers in 8,500 ha were separated during this 10-year, $332 million program.

Groundwater Impacts from Stormwater Infiltration

Prior to urbanization, groundwater recharge results from infiltration of precipitation through pervious surfaces, including grasslands and woods. This infiltrating water is relatively uncontaminated. With urbanization, the permeable soil surface area through which recharge by infiltration could occur is reduced. This results in much less groundwater recharge and greatly increased surface runoff. In addition, the waters available for recharge generally carry increased quantities of pollutants. With urbanization, new problematic sources of groundwater recharge also occur, including recharge from domestic septic tanks, percolation basins and industrial waste injection wells, and from agricultural and residential irrigation.

The following paragraphs (from Pitt et al. 1994 and 1996) describe the stormwater pollutants that have the greatest potential of adversely affecting groundwater quality during inadvertent or intentional stormwater infiltration. Also included are suggestions on ways to minimize these potential problems.

Constituents of Concern

Nutrients

Nitrates are one of the most frequently encountered contaminants in groundwater. Groundwater contamination of phosphorus has not been as widespread, or as severe, as for nitrogen compounds. Whenever nitrogen-containing compounds come into contact with soil, a potential for nitrate leaching into groundwater exists, especially in rapid-infiltration wastewater basins, stormwater infiltration devices, and in agricultural areas. Nitrate has leached from fertilizers and affected groundwaters under various turf grasses in urban areas, including golf courses, parks and home lawns. Significant leaching of nitrates occurs during the cool, wet seasons. Cool temperatures reduce denitrification and ammonia volatilization, and limit microbial nitrogen immobilization and plant uptake.

The use of slow-release fertilizers is recommended in areas having potential groundwater nitrate problems. The slow-release fertilizers include urea formaldehyde (UF), methylene urea, isobutylidene diurea (IBDU), and sulfur-coated urea. Residual nitrate concentrations are highly variable in soil due to soil texture, mineralization, rainfall and irrigation patterns, organic matter content, crop yield, nitrogen fertilizer/sludge rate, denitrification, and soil compaction. Nitrate is highly soluble (>1 kg/l) and will stay in solution in the percolation water, after leaving the root zone, until it reaches the groundwater.

Pesticides

Urban pesticide contamination of groundwater can result from municipal and homeowner use of pesticides for pest control and their subsequent collection in stormwater runoff. Pesticides that have been found in urban groundwaters include: 2,4-D, 2,4,5-T, atrazine, chlordane, diazinon, ethion, malathion, methyl trithion, silvex, and simazine. Heavy repetitive use of mobile pesticides on irrigated and sandy soils likely contaminates groundwater. Fungicides and nematocides must be mobile in order to reach the target pest and hence, they generally have the highest contamination potential. Pesticide leaching depends on patterns of use, soil texture, total organic carbon content of the soil, pesticide persistence, and depth to the water table.

The greatest pesticide mobility occurs in areas with coarse-grained or sandy soils without a hardpan layer, having low clay and organic matter content and high permeability. Structural voids, which are generally found in the surface layer of finer-textured soils rich in clay, can transmit pesticides rapidly when the voids are filled with water and the adsorbing surfaces of the soil matrix are bypassed. In general, pesticides with low water solubilities, high octanol-water partitioning coefficients, and high carbon partitioning coefficients are less mobile. The slower moving pesticides have been recommended in areas of groundwater contamination concern. These include the fungicides iprodione and triadimefon, the insecticides isofenphos and chlorpyrifos and the herbicide glyphosate. The most mobile pesticides include: 2,4-D, acenaphthylene, alachlor, atrazine, cyanazine, dacthal, diazinon, dicamba, malathion, and metolachlor.

Pesticides decompose in soil and water, but the total decomposition time can range from days to years. Literature half-lives for pesticides generally apply to surface soils and do not account for the reduced microbial activity found deep in the vadose zone. Pesticides with a 30 day half life can show considerable leaching. An order-of-magnitude difference in half-life results in a five- to ten-fold difference in percolation loss. Organophosphate pesticides are less persistent than organochlorine pesticides, but they also are not strongly adsorbed by the sediment and are likely to leach into the vadose zone, and the groundwater.

Other Organics

The most commonly occurring organic compounds that have been found in urban groundwaters include phthalate esters (especially bis(2-ethylhexyl)phthalate) and phenolic compounds. Other organics more rarely found, possibly due to losses during sample collection, have included the volatiles: benzene, chloroform, methylene chloride, trichloroethylene, tetrachloroethylene, toluene, and xylene. PAHs (especially benzo(a)anthracene, chrysene, anthracene and benzo(b)fluoroanthenene) have also been found in groundwaters near industrial sites.

Groundwater contamination from organics, like from other pollutants, occurs more readily in areas with sandy soils and where the water table is near the land surface. Removal of organics from the soil and recharge water can occur by one of three methods: volatilization, sorption, and degradation. Volatilization can significantly reduce the concentrations of the most volatile compounds in groundwater, but the rate of gas transfer from the soil to the air is usually limited by the presence of soil water. Hydrophobic sorption onto soil organic matter limits the mobility of less soluble base/neutral and acid extractable compounds through organic soils and the vadose zone. Sorption is not always a permanent removal mechanism, however. Organic re-solubilization can occur during wet periods following dry periods. Many organics can be at least partially degraded by microorganisms, but others cannot. Temperature, pH, moisture content, ion exchange capacity of soil, and air availability may limit the microbial degradation potential for even the most degradable organic.

Pathogenic Microorganisms

Viruses have been detected in groundwater where stormwater recharge basins were located short distances above the aquifer. Enteric viruses are more resistant to environmental factors than enteric bacteria and they exhibit longer survival times in natural waters. They can occur in potable and marine waters in the absence of fecal coliforms. Enteroviruses are also more resistant to commonly used disinfectants than are indicator bacteria, and can occur in groundwater in the absence of indicator bacteria.

The factors that affect the survival of enteric bacteria and viruses in the soil include pH, antagonism from soil microflora, moisture content, temperature, sunlight, and organic matter. The two most important attributes of viruses that permit their long-term survival in the environment are their structure and very small size. These characteristics permit virus occlusion and protection within colloid-size particles. Viral adsorption is promoted by increasing cation concentration, decreasing pH and decreasing soluble organics. Since the movement of viruses through soil to groundwater occurs in the liquid phase and involves water movement and associated suspended virus particles, the distribution of viruses between the adsorbed and liquid phases determines the viral mass available for movement. Once the virus reaches the groundwater, it can travel laterally through the aquifer until it is either adsorbed or inactivated.

The major bacterial removal mechanisms in soil are straining at the soil surface and at intergrain contacts, sedimentation, sorption by soil particles, and inactivation. Because of their larger size than for viruses, most bacteria are, therefore, retained near the soil surface due to this straining effect. In general, enteric bacteria survive in soil between two and three months, although survival times up to five years have been documented.

Heavy Metals and Other Inorganic Compounds

Heavy metals and other inorganic compounds in stormwater of most environmental concern, from a groundwater pollution standpoint, are aluminum, arsenic, cadmium, chromium, copper, iron, lead, mercury, nickel, and zinc. However, the majority of these compounds, with the consistent exception of zinc, are mostly found associated with the particulate solids in stormwaters and are thus relatively easily removed through sedimentation practices. Filterable forms of the metals may also be removed by either sediment adsorption or are organically complexed with other particulates.

In general, studies of recharge basins receiving large metal loads found that most of the heavy metals are removed either in the basin sediment or in the vadose zone. Dissolved metal ions are removed from stormwater during infiltration mostly by adsorption onto the near-surface particles in the vadose zone, while the particulate metals are filtered out at the soil surface. Studies at recharge basins found that lead, zinc, cadmium, and copper accumulated at the soil surface with little downward movement over many years. However, nickel, chromium, and zinc concentrations have exceeded regulatory limits in the soils below a recharge area at a commercial site. Elevated groundwater heavy metal concentrations of aluminum, cadmium, copper, chromium, lead, and zinc have been found below stormwater infiltration devices where the groundwater pH has been acidic. Allowing percolation ponds to go dry between storms can be counterproductive to the removal of lead from the water during recharge. Apparently, the adsorption bonds between the sediment and the metals can be weakened during the drying period.

Similarities in water quality between runoff water and groundwater has shown that there is significant downward movement of copper and iron in sandy and loamy soils. However, arsenic, nickel, and lead did not significantly move downward through the soil to the groundwater. The exception to this was some downward movement of lead with the percolation water in sandy soils beneath stormwater recharge basins. Zinc, which is more soluble than iron, has been found in higher concentrations in groundwater than iron. The order of attenuation in the vadose zone from infiltrating stormwater is: zinc (most mobile) > lead > cadmium > manganese > copper > iron > chromium > nickel > aluminum (least mobile).

Salts

Salt applications for winter traffic safety is a common practice in many northern areas and the sodium and chloride, which are collected in the snowmelt, travel down through the vadose zone to the groundwater with little attenuation. Soil is not very effective at removing salts. Salts that are still in the percolation water after it travels through the vadose zone will contaminate the groundwater. Infiltration of stormwater has led to increases in sodium and chloride concentrations above background concentrations. Fertilizer and pesticide salts also accumulate in urban areas and can leach through the soil to the groundwater.

Studies of depth of pollutant penetration in soil have shown that sulfate and potassium concentrations decrease with depth, while sodium, calcium, bicarbonate, and chloride concentrations increase with depth. Once contamination with salts begin, the movement of salts into the groundwater can be rapid. The salt concentration may not decrease until the source of the salts is removed.

Recommendations to Protect Groundwater During Stormwater Infiltration

Table 5-1 is a summary of the pollutants found in stormwater that may cause groundwater contamination problems for various reasons. This table does not consider the risk associated with using groundwater contaminated with these pollutants. Characteristics of concern include high mobility (low sorption potential) in the vadose zone, high abundance (high concentrations and high detection frequencies) in stormwater, and high soluble fractions (small fraction associated with particulates which would have little removal potential using conventional stormwater sedimentation controls) in the stormwater.

The contamination potential is the lowest rating of the influencing factors. As an example, if no pretreatment was to be used before percolation through surface soils, the mobility and abundance criteria are most important. If a compound was mobile, but was in low abundance (such as for VOCs), then the groundwater contamination potential would be low. However, if the compound was mobile and was also in high abundance (such as for sodium chloride, in certain conditions), then the groundwater contamination would be high.

If sedimentation pretreatment was to be used before infiltration, then much of the pollutants will likely be removed before infiltration. In this case, all three influencing factors (mobility, abundance in stormwater, and soluble fraction) would be considered important. As an example, chlordane would have a low contamination potential with sedimentation pretreatment, while it would have a moderate contamination potential if no pretreatment was used. In addition, if subsurface infiltration/injection was used instead of surface percolation, the compounds would most likely be more mobile, making the abundance criteria the most important, with some regard given to the filterable fraction information for operational considerations.

Table 5-1 is only appropriate for initial estimates of contamination potential because of the simplifying assumptions made, such as the likely worst case mobility measures for sandy soils having low organic content. If the soil was clayey and had a high organic content, then most of the organic compounds would be less mobile than shown on this table. The abundance and filterable fraction information is generally applicable for warm weather stormwater runoff at residential and commercial area outfalls. The concentrations and detection frequencies would likely be greater for critical source areas (especially vehicle service areas) and critical land uses (especially manufacturing industrial areas).

Table 5-1. Groundwater contamination potential for stormwater pollutants (Pitt et al. 1996).

Categories	Compounds	Mobility (sandy/low organic soils)	Abundance in storm-water	Fraction filterable	Contamination potential for surface infilt. and no pretreatment	Contamination potential for surface infilt. with sediment-ation	Contamination potential for sub-surface inj. with minimal pretreatment
Nutrients	Nitrates	mobile	low/moderate	high	low/moderate	low/moderate	low/moderate
Pesticides	2,4-D	mobile	low	likely low	low	low	low
	γ-BHC (lindane)	intermediate	moderate	likely low	moderate	low	moderate
	malathion	mobile	low	likely low	low	low	low
	atrazine	mobile	low	likely low	low	low	low
	chlordane	intermediate	moderate	very low	moderate	low	moderate
	diazinon	mobile	low	likely low	low	low	low
Other organics	VOCs	mobile	low	very high	low	low	low
	1,3-dichloro-benzene	low	high	high	low	low	high
	anthracene	intermediate	low	moderate	low	low	low
	benzo(a) anthracene	intermediate	moderate	very low	moderate	low	moderate
	bis (2-ethylhexyl) phthalate	intermediate	moderate	likely low	moderate	low?	moderate
	butyl benzyl phthalate	low	low/moderate	moderate	low	low	low/moderate
	fluoranthene	intermediate	high	high	moderate	moderate	high
	fluorene	intermediate	low	likely low	low	low	low
	naphthalene	low/inter.	low	moderate	low	low	low
	penta-chlorophenol	intermediate	moderate	likely low	moderate	low?	moderate
	phenanthrene	intermediate	moderate	very low	moderate	low	moderate
	pyrene	intermediate	high	high	moderate	moderate	high
Pathogens	enteroviruses	mobile	likely present	high	high	high	high
	Shigella	low/inter.	likely present	moderate	low/moderate	low/moderate	high
	Pseudomonas aeruginosa	low/inter.	very high	moderate	low/moderate	low/moderate	high
	protozoa	low/inter.	likely present	moderate	low/moderate	low/moderate	high
Heavy metals	nickel	low	high	low	low	low	high
	cadmium	low	low	moderate	low	low	low
	chromium	inter./very low	moderate	very low	low/moderate	low	moderate
	lead	very low	moderate	very low	low	low	moderate
	zinc	low/very low	high	high	low	low	high
Salts	chloride	mobile	seasonally high	high	high	high	high

The stormwater pollutants of most concern (those that may have the greatest adverse impacts on groundwaters) include:

1. Nutrients: nitrate has a low to moderate groundwater contamination potential for both surface percolation and subsurface infiltration/injection practices because of its relatively low concentrations found in most stormwaters. However, if the stormwater nitrate concentration was high, then the groundwater contamination potential would also likely be high.

2. Pesticides: lindane and chlordane have moderate groundwater contamination potentials for surface percolation practices (with no pretreatment) and for subsurface injection (with minimal pretreatment). The groundwater contamination potentials for both of these compounds would likely be substantially reduced with adequate sedimentation pretreatment. Pesticides have been mostly found in urban runoff from residential areas, especially in dry-weather flows associated with landscaping irrigation runoff.

3. Other organics: 1,3-dichlorobenzene may have a high groundwater contamination potential for subsurface infiltration/injection (with minimal pretreatment). However, it would likely have a lower groundwater contamination potential for most surface percolation practices because of its relatively strong sorption to vadose zone soils. Both pyrene and fluoranthene would also likely have high groundwater contamination potentials for subsurface infiltration/injection practices, but lower contamination potentials for surface percolation practices because of their more limited mobility through the unsaturated zone (vadose zone). Others (including benzo(a)anthracene, bis (2-ethylhexyl) phthalate, pentachlorophenol, and phenanthrene) may also have moderate groundwater contamination potentials, if surface percolation with no pretreatment, or subsurface injection/infiltration is used. These compounds would have low groundwater contamination potentials if surface infiltration was used with sedimentation pretreatment. Volatile organic compounds (VOCs) may also have high groundwater contamination potentials if present in the stormwater (likely for some industrial and commercial facilities and vehicle service establishments). The other organics, especially the volatiles, are mostly found in industrial areas. The phthalates are found in all areas. The PAHs are also found in runoff from all areas, but they are in higher concentrations and occur more frequently in industrial areas.

4. Pathogens: enteroviruses likely have a high groundwater contamination potential for all percolation practices and subsurface infiltration/injection practices, depending on their presence in stormwater (likely if contaminated with sanitary sewage). Other pathogens, including Shigella, Pseudomonas aeruginosa, and various protozoa, would also have high groundwater contamination potentials if subsurface infiltration/injection practices are used without disinfection. If disinfection (especially by chlorine or ozone) is used,

then disinfection byproducts (such as trihalomethanes or ozonated bromides) would have high groundwater contamination potentials. Pathogens are most likely associated with sanitary sewage contamination of storm drainage systems, but several bacterial pathogens are commonly found in surface runoff in residential areas.

5. Heavy metals: nickel and zinc would likely have high groundwater contamination potentials if subsurface infiltration/injection was used. Chromium and lead would have moderate groundwater contamination potentials for subsurface infiltration/injection practices. All metals would likely have low groundwater contamination potentials if surface infiltration was used with sedimentation pretreatment. Zinc is mostly found in roof runoff and other areas where galvanized metal comes into contact with rainwater.

6. Salts: chloride would likely have a high groundwater contamination potential in northern areas where road salts are used for traffic safety, irrespective of the pretreatment, infiltration or percolation practice used. Salts are at their greatest concentrations in snowmelt and early spring runoff in northern areas.

It has been suggested that, with a reasonable degree of site-specific design considerations to compensate for soil characteristics, infiltration can be very effective in controlling both urban runoff quality and quantity problems (EPA 1983). This strategy encourages infiltration of urban runoff to replace the natural infiltration capacity lost through urbanization and to use the natural filtering and sorption capacity of soils to remove pollutants.

However, potential groundwater contamination through infiltration of some types of urban runoff requires some restrictions. Infiltration of urban runoff having potentially high concentrations of pollutants that may pollute groundwater requires adequate pretreatment, or the diversion of these waters away from infiltration devices. The following general guidelines for the infiltration of stormwater and other storm drainage effluent are recommended in the absence of comprehensive site-specific evaluations:

1. Dry-weather storm drainage effluent should be diverted from infiltration devices because of their probable high concentrations of soluble heavy metals, pesticides, and pathogenic microorganisms.

2. Combined sewage overflows should be diverted from infiltration devices because of their poor water quality, especially high pathogenic microorganism concentrations, and high clogging potential.

3. Snowmelt runoff should also be diverted from infiltration devices because of its potential for having high concentrations of soluble salts.

4. Runoff from manufacturing industrial areas should also be diverted from infiltration devices because of its potential for having high concentrations of soluble toxicants.

5. Construction site runoff must be diverted from stormwater infiltration devices (especially subsurface devices) because of its high SS concentrations, which would quickly clog infiltration devices.

6. Runoff from other critical source areas, such as vehicle service facilities and large parking areas, should at least receive adequate pretreatment to eliminate their groundwater contamination potential before infiltration.

7. Runoff from residential areas (the largest component of urban runoff from most cities) is generally the least polluted urban runoff flow and should be considered for infiltration. Very little treatment of residential area stormwater runoff should be needed before infiltration, especially if surface infiltration is through the use of grass swales. If subsurface infiltration (e.g., French drains, infiltration trenches, dry wells) is used, then some pretreatment may be needed, such as by using grass filter strips, or other surface filtration devices.

All other runoff should include pretreatment using sedimentation processes before infiltration, to both minimize groundwater contamination and to prolong the life of the infiltration device (if needed). This pretreatment can take the form of approaches such as grass filters, sediment sumps, and wet detention ponds depending on the runoff volume to be treated and other site specific factors. Pollution prevention can also play an important role in minimizing groundwater contamination problems, including reducing the use of galvanized metals, pesticides, and fertilizers in critical areas. The use of specialized treatment devices can also play an important role in treating runoff from critical source areas before these more contaminated flows commingle with cleaner runoff from other areas. Sophisticated treatment schemes, especially the use of chemical processes or disinfection, may not be warranted, except in special cases, especially considering the potential of forming harmful treatment by-products (such as THMs and soluble aluminum).

Most past stormwater quality monitoring has not been adequate to completely evaluate groundwater contamination potential. The following list shows the parameters that are recommended to be monitored if stormwater contamination potential needs to be considered, or infiltration devices are to be used. Other analyses are appropriate for additional monitoring objectives (such as evaluating surface water problems). In addition, all phases of urban runoff should be sampled, including stormwater runoff, dry-weather flows, and snowmelt.

- Contamination potential:
  - Nutrients (especially nitrates)
  - Salts (especially chloride)

  - VOCs (if expected in the runoff, such as from manufacturing industrial or vehicle service areas, could screen for VOCs with purgable organic carbon, POC, analyses)
  - Pathogens (especially enteroviruses, if possible, along with other pathogens such as Pseudomonas aeruginosa, Shigella, and pathogenic protozoa)
  - Bromide and total organic carbon, TOC (to estimate disinfection by-product generation potential, if disinfection by either chlorination or ozone is being considered)
  - Pesticides, in both filterable and total sample components (especially lindane and chlordane)
  - Other organics, in both filterable and total sample components (especially 1,3 dichlorobenzene, pyrene, fluoranthene, benzo (a) anthracene, bis (2-ethylhexyl) phthalate, pentachlorophenol, and phenanthrene)
  - Heavy metals, in both filterable and total sample components (especially chromium, lead, nickel, and zinc)

- Operational considerations:
  - Sodium, calcium, and magnesium (in order to calculate the sodium adsorption ratio to predict clogging of clay soils)
  - Suspended solids (to determine the need for sedimentation pretreatment to prevent clogging)

The Technical University of Denmark (Mikkelsen et al. 1996a and 1996b) has been involved in a series of tests to examine the effects of stormwater infiltration on soil and groundwater quality. They found that heavy metals and PAHs present little groundwater contamination threat, if surface infiltration systems are used. However, they express concern about pesticides, which are much more mobile. Squillace et al. (1996) along with Zogorski et al. (1996) presented information concerning stormwater and its potential as a source of groundwater MTBE contamination. Mull (1996) stated that traffic areas are the third most important source of groundwater contamination in Germany (after abandoned industrial sites and leaky sewers). The most important contaminants are chlorinated hydrocarbons, sulfate, organic compounds, and nitrates. Heavy metals are generally not an important groundwater contaminant because of their affinity for soils. Trauth and Xanthopoulus (1996) examined the long-term trends in groundwater quality at Karlsruhe, Germany. They found that the urban landuse is having a long-term influence on the groundwater quality. The concentration of many pollutants have increased by about 30 to 40% over 20 years. Hütter and Remmler (1996) describe a groundwater monitoring plan, including monitoring wells that were established during the construction of an infiltration trench for stormwater disposal in Dortmund, Germany. The worst case problem expected is with zinc, if the infiltration water has a pH value of 4.

References

Allen, H. E., editor (1996). Metal Contaminated Aquatic Sediments. Ann Arbor Press. Chelsea, MI. 350 pgs.

Benke, A.C., G.E. Willeke, F.K. Parrish and D.L. Stites (1981). Effects of Urbanization on Stream Ecosystems. School of Biology. Environmental Resources Center. Report No. ERC 07-81. Georgia Institute of Technology. Atlanta, GA.

Burton, G.A. and R.E. Pitt (to be published in 1998). A Manual for Conducting Evaluations of the Effects of Urban Runoff on Aquatic Life. CRC Press.

Callahan, M.A., M.W. Slimak, N.W. Gabel, I.P. May, C.F. Fowler, J.R. Freed, P. Jennings, R.L. Durfee, F.C. Whitmore, B. Maestri, W.R. Mabey, B.R. Holt, and C. Gould (1979). Water Related Environmental Fates of 129 Priority Pollutants. U.S. Environmental Protection Agency. Monitoring and Data Support Division. EPA-4-79-029a and b. Washington D.C.

Claytor, R.A. and W. Brown (1996). Environmental Indicators to Assess the Effectiveness of Municipal and Industrial Stormwater Control Programs. Prepared for the U.S. EPA, Office of Wastewater Management. Center for Watershed Protection. Silver Spring, MD. 210 pgs.

Claytor, R.A (1996a). An introduction to stormwater indicators: an urban runoff assessment tool. Watershed Protection Techniques. Vol. 2, no. 2, pp. 321 - 328. Spring.

Claytor, R.A (1996b). Habitat and biological monitoring reveals headwater stream impairment n Delaware's Piedmont. Watershed Protection Techniques. Vol. 2, no. 2, pp. 358 - 360. Spring.

Cook, W.L., F. Parrish, J.D. Satterfield, W.G. Nolan and P.E. Gaffney (1983). Biological and Chemical Assessment of Nonpoint Source Pollution in Georgia: Ridge-Valley and Sea Island Streams. Department of Biology. Georgia State University. Atlanta, GA.

Crunkilton, R., J. Kleist, J. Ramcheck, W. DeVita, and D. Villeneueve (1997). Assessment of the response of aquatic organisms to long-term in-situ exposures to urban runoff. Presented at the Effects of Watershed Developments and Management on Aquatic Ecosystems conference. Snowbird, UT. August 4 – 9, 1996. Edited by L.A. Roesner. ASCE. New York, NY.

CTA, Inc. (1983). Georgia Nonpoint Source Impact Assessment Study: Blue Ridge/Upland Georgia Cluster, Piedmont Cluster, and Gulf Coastal Plain Cluster. Georgia Environmental Protection Division. Dept. of Natural Resources. Atlanta, GA.

Davies, P.H. (1986). Toxicology and chemistry of metals in urban runoff. In: Urban Runoff Quality: Impact and Quality Enhancement Technology. Engineering Foundation Conference. Henniker, NH. ASCE. New York, NY.

DePinto, J.V., T.C. Young and S.C. Martin (1980). Aquatic sediments. Journal of Water Pollution Control Federation. Vol. 52. No. 6. pp 1656-70. June.

Dyer, S.D. and C.E. White (1996). A watershed approach to assess mixture toxicity via integration of public and private databases. Abstract Book: SETAC 17th Annual Meeting. pg. 96. Washington, D.C. Nov. 17 – 21.

Ebbert, J.C., J. E. Poole, and K.L. Payne (1983). Data collected by the U.S. Geological Survey during a study of urban runoff in Bellevue, WA. 1979-82. Preliminary U.S. Geological Survey Open-File Report. Tacoma, WA.

Ehrenfeld, J.G. and J.P. Schneider (1983). The Sensitivity of Cedar Swamps to the Effects of Non-Point Pollution Associated with Suburbanization in the New Jersey Pine Barrens. U.S. Environmental Protection Agency. Office of Water Policy. PB8-4-136779. Washington, D.C. September.

Environmental Science & Technology (1996a). Toxicity of aquatic mixtures yielding to new theoretical approach. Vol. 30, no. 4, pp. 155a - 156a. April.

Environmental Science & Technology (1986b). News Briefs. Vol. 30, no. 7, pg. 290a. July.

EPA (U.S. Environmental Protection Agency) (1983). Results of the Nationwide Urban Runoff Program. Water Planning Division. PB 84-185552. Washington, D.C. December.

Field, R., and R. Pitt (1990). Urban storm-induced discharge impacts: US Environmental Protection Agency research program review. Water Science and Technology. Vol. 22, No. 10/11.

Garie, H.L. and A. McIntosh (1986). Distribution of benthic macroinvertebrates in a stream exposed to urban runoff. Water Resources Bulletin. Vol. 22. No. 3. pp. 447-455.

Garie, H.L. and A. McIntosh (1990). Distribution of benthic macroinvertebrates in a stream exposed to urban runoff. Water Science and Technology. Vol. 22, No. 10/11.

Haile and the Santa Monica Bay Restoration Project (1996). An Epidemiological Study of Possible Adverse Health Effects of Swimming in Santa Monica Bay. Santa Monica Bay Restoration Project. Monterey Park, CA. October.

Herricks. E.E., editor (1995). Stormwater Runoff and Receiving Systems: Impact, Monitoring and Assessment. Conference of the Engineering Foundation/ASCE held in 1991 in Mt. Crested Butte, CO. Lewis/CRC Press. Boca Raton. 458 pgs.

Herricks, E.E, I. Milne, and I. Johnson (1996). A protocol for wet weather discharge toxicity assessment. Volume 4, pg. 13 - 24. WEFTEC'96: Proceedings of the 69th Annual Conference & Exposition. Dallas, TX.

Hütter, U. and F. Remmler (1996). Stormwater infiltration at a site with critical subsoil conditions: Investigations of soil, seepage water and groundwater. 7th International Conference on Urban Drainage. Hannover, Germany. Edited by F. Sieker and H-R. Verworn. International Association on Water Quality, London. pp. 713 – 718 . Sept. 9 – 13.

Ireland, D.S., G.A. Burton, Jr., and G.G. Hess (1996). In-situ toxicity evaluations of turbidity and photoinduction of polycyclic aromatic hydrocarbons. Environmental Toxicology and Chemistry. Vol. 15, no. 4, pp. 574 - 581. April.

Johnson, I., E.E. Herricks, and I. Milne (1996). Application of a test battery for wet weather discharge toxicity analyses. Volume 4, pg. 219 - 229. WEFTEC'96: Proceedings of the 69th Annual Conference & Exposition. Dallas, TX.

Klein, R.D (1979). Urbanization and stream quality impairment. Water Resources Bulletin. Vol. 15. No. 4. August.

Lee, G.F. and A. Jones-Lee (1993). Water quality impacts of stormwater-associated contaminants: focus on real problems. Water Science and Technology. Vol. 28, No. 3-5, pp. 231-240.

Lee, G.F. and A. Jones-Lee (1995a). Deficiencies in stormwater quality monitoring. in: Stormwater NPDES Related Monitoring Needs. Edited by H.C. Torno. Proceedings of an Engineering Foundation Conference. Mt. Crested Butte, CO. August 1994. ASCE. New York, NY.

Lee, G.F. and A. Jones-Lee (1995b). Issues in managing urban stormwater runoff quality. Water/Engineering Management. Vol. 142, No. 5. pp. 51-53. May.

Lenet, D.R., D.L. Penrose, and K. Eagleson 1979). Biological Evaluation of Non-Point Sources of Pollutants in North Carolina Streams and Rivers. North Carolina Division of Environmental Management, Biological Series #102. North Carolina Dept. of Natural Resources and Community Development. Raleigh, NC.

Lenet, D. and K. Eagleson (1981). Ecological Effects of Urban Runoff on North Carolina Streams. North Carolina Division of Environmental Management, Biological Series #104. North Carolina Dept. of Natural Resources and Community Development. Raleigh, NC.

Lenet, D.R., D.L. Penrose, and K.W. Eagleson (1981). Variable effects of sediment addition on stream benthos. Hydrobiologia. Vol. 79. pp. 187-194.

Mancini, J. and A. Plummer (1986). Urban runoff and water quality criteria. In: Urban Runoff Quality – Impact and Quality Enhancement Technology. Edited by B. Urbonas and L.A. Roesner. Engineering Foundation Conference. Henniker, Hew Hampshire. ASCE. NY. pp. 133-149. June.

Marcy, S. and J. Gerritsen (1996). Developing diverse assessment endpoints to address multiple stressors in watershed ecological risk assessment. Abstract Book: SETAC 17th Annual Meeting. pg. 96. Washington, D.C. Nov. 17 - 21.

Medeiros, C. and R.A. Coler (1982). A Laboratory/Field Investigation into the Biological Effects of Urban Runoff. Water Resources Research Center. University of Massachusetts. Amherst, MA. July.

Medeiros, C., R.A. Coler, and E.J. Calabrese (1984). A laboratory assessment of the toxicity of urban runoff on the fathead minnow (Pimephales promelas). Journal of Environmental Science Health. Vol. A19. No. 7. pp. 847-861.

Mikkelsen, P.S., H. Madsen, H. Rosgjerg, and P. Harremoës (1996a). Properties of extreme point rainfall III: Identification of spatial inter-site correlation structure. Atmospheric Research.

Mikkelsen, P.S., K. Arngjerg-Nielsen, and P. Harremoës (1996b). Consequences for established design practice from geographical variation of historical rainfall data. Proceedings: 7th International Conference on Urban Storm Drainage. Hannover, Germany. Sept. 9 – 13.

Mote Marine Laboratory (1984). Biological and Chemical Studies on the Impact of Stormwater Runoff upon the Biological Community of the Hillsborough River, Tampa, FL. Stormwater Management Division, Dept. of Public Works. Tampa, FL. March.

Mull, R. (1996). Water exchange between leaky sewers and aquifers. 7th International Conference on Urban Storm Drainage. Hannover, Germany. Sept. 9-13, 1996. Edited by F. Sieker and H-R. Verworn. IAHR/IAWQ. SuG-Verlagsgesellschaft. Hannover, Germany. pp. 695-700.

Parmer, K.D. (1993). Photo and Biodegradation of Pyrene and Benzo(a)pyrene in a Model of the Near Surface Environment. Ph.D. dissertation. Department of Environmental Health Science. The University of Alabama at Birmingham. 299 pgs.

Pedersen, Edward Robert (1981). The Use of Benthic Invertebrate Data for Evaluating Impacts of Urban Stormwater Runoff. Masters thesis submitted to the College of Engineering. University of Washington. Seattle, WA.

Perkins, M. A. (1982). An Evaluation of Instream Ecological Effects Associated with Urban Runoff to a Lowland Stream in Western Washington. U.S. Environmental Protection Agency. Corvallis Environmental Research Laboratory. Corvallis, OR. July.

Pitt, R. and M. Bozeman (1982). Sources of Urban Runoff Pollution and Its Effects on an Urban Creek. EPA-600/52-82-090. U.S. Environmental Protection Agency. Cincinnati, OH. December.

Pitt, R.E., and P. Bissonnette (1983). Bellevue Urban Runoff Program, Summary Report. PB84 237213. Water Planning Division. U.S. Environmental Protection Agency. Washington, D.C. December.

Pitt, R. E. (1991). Biological effects of urban runoff discharges. in: Effects of Urban Runoff on Receiving Systems: An Interdisciplinary Analysis of Impact, Monitoring, and Management, Engineering Foundation Conference. Mt. Crested Butte, CO. ASCE. New York, NY.

Pitt, R., S. Clark, and K. Parmer (1994). Protection of Groundwater from Intentional and Nonintentional Stormwater Infiltration. U.S. Environmental Protection Agency. EPA/600/SR-94/051. PB94-165354AS. Storm and Combined Sewer Program. Cincinnati, Ohio. 187 pgs. May.

Pitt, R. (1995). Effects of urban runoff on aquatic biota. In: Handbook of Ecotoxicology (Edited by D.J. Hoffman, B.A. Rattner, G.A. Burton, Jr. and J.Cairns, Jr.). Lewis Publishers/CRC Press. Boca Raton, FL. pp. 609-630.

Pitt, R., S. Clark, K. Parmer, and R. Field (1996). Groundwater Contamination from Stormwater Infiltration. Ann Arbor Press. Chelsea, MI. 218 pages.

Pratt, J.M., R.A. Coler and P.J. Godfrey (1981). Ecological effects of urban stormwater runoff on benthic macroinvertibrates inhabiting the Green River, Massachusetts. Hydrobiologia. Vol. 83. pp. 29-42.

Prych, Edmund A. and J.C. Ebbert (undated). Quantity and Quality of Storm Runoff from Three Urban Catchments in Bellevue, WA. Preliminary U.S. Geological Survey Water Resources Investigations Report. Tacoma, WA.

Rainbow, P.S. (1996). Chapter 18: Heavy metals in aquatic invertebrates. In: Environmental Contaminants in Wildlife: Interpreting Tissue Concentrations. Edited by W.N. Beyer, G.H. Heinz, and A.W. Redmon-Norwood. CRC/Lewis Press. Boca Raton, FL. pp. 405 - 425.

Richey, Joanne Sloane (1982). Effects of Urbanization on a Lowland Stream in Western Washington. Doctor of Philosophy dissertation. University of Washington. Seattle, WA.

Richey, Joanne Sloane, Michael A. Perkins, and Kenneth W. Malueg (1981). The effects of urbanization and stormwater runoff on the food quality in two salmonid streams. Verh. Internat. Werein. Limnol. Vol. 21, Pages 812-818. Stuttgart, Germany. October.

Schueler, T. (editor) (1996). Stream channel geometry used to assess land use impacts in the Pacific Northwest. Watershed Protection Techniques. Vol. 2, no. 2, pp. 345 - 348. Spring.

Scott, J.B., C.R. Steward, and Q.J. Stober (1982). Impacts of Urban Runoff on Fish Populations in Kelsey Creek, WA. Contract No. R806387020. U.S. Environmental Protection Agency. Corvallis Environmental Research Laboratory. Corvallis, OR. May.

Squillace, P.J., J.S. Zogorski, W.G. Wilber, and C.V. Price (1996). Preliminary assessment of the occurrence and possible sources of MTBE in groundwater in the United States, 1993 - 94. Environmental Science & Technology. Vol. 30, no. 5, pp. 1721 - 1730. May.

Stephenson, D. (1996). Evaluation of effects of urbanization on storm runoff. 7th International Conference on Urban Storm Drainage. Hannover, Germany. Sept. 9-13, 1996. Edited by F. Sieker and H-R. Verworn. IAHR/IAWQ. SuG-Verlagsgesellschaft. Hannover, Germany. pp. 31-36.

Striegl, R.G. (1996). Effects of stormwater runoff on an urban lake, Lake Ellyn at Glen Ellyn, IL. USGS open file report 84-603. Lakewood, CO.

Trauth, R. and C. Xanthopoulos (1996). Non-point pollution of groundwater in urban areas. 7th International Conference on Urban Drainage. Hannover, Germany. Edited by F. Sieker and H-R. Verworn. International Association on Water Quality, London. pp. 701-706. Sept. 9 – 13.

Water Environment & Technology (1996a). News Watch: Sewer separation lowers fecal coliform levels in the Mississippi River. Vol. 8, no. 11, pp. 21 - 22. November.

Water Environment & Technology (1996b). Research Notes: Beachgoers at Risk from Urban Runoff. Vol. 8, no. 11, pg. 65. November.

Wilber, W. G., and J. V. Hunter (1980). The influence of urbanization on the transport of heavy metals in New Jersey Streams. Water Resources Research Institute. Rutgers University. New Brunswick, NJ.

Zogorski, J.S., A.B. Morduchowitz, A.L. Baehr, B.J. Bauman, D.L. Conrad, R.T. Drew, N.E. Korte, W.W. Lapham, J.F. Pankow, and E.R. Washington (1996). Fuel Oxygenates and Water Quality: Current Understanding of Sources, Occurrence in Natural Waters, Environmental Behavior, Fate, and Significance. Office of Science and Technology. Washington, D.C.

Chapter 6

Collection Systems

James P. Heaney, Len Wright, and David Sample

Introduction

Stormwater and wastewater collection systems are a critical link in the urban water cycle, especially under wet-weather conditions. In the context of pollution control, these systems transport sanitary wastewater, stormwater, industrial wastewater, non-point source pollution, and inflow/infiltration (I/I).

Research in the area of collection systems as a means of wet-weather pollution control is showing signs of renewed activity, especially in Europe and Japan (Henze et al. (1997), Sieker and Verworn (Ed.) 1996, Ashley (Ed.) 1996, Bally et al. (Ed.) 1996). Case studies of recent applications of innovations in this country are also receiving attention, as evidenced by recent Water Environment Federation technical conferences (WEF 1994a, 1994b, 1995a, 1995b, 1996) and a recent EPA seminar (USEPA 1996b). By applying new technology and revisiting traditional urban water problems with a fresh outlook, advances are being made in a wide variety of sewer related areas. By reviewing successful applications of research in recent projects, a vision of successful wet-weather management of collection systems of the future may be formulated.

An historical review of collection systems in the U.S. helps with understanding the problems associated with modern sewer collection systems. Many of the early sewers, including some from before the turn of the century, are still in service. As cities grew, the need for stormwater and wastewater conveyance became a necessity to protect human health. Stormwater and sanitary waste were generally conveyed to the nearest natural water body. In fact, the modern word “sewer” is derived from the old English word meaning “seaward” (Gayman 1996).

In the late nineteenth and early part of the twentieth century, these conveyance systems were “intercepted” into a smaller conveyance sized to accommodate a multiple of the estimated dry weather sanitary flow (Moffa 1990, Foil et al. 1993, Metcalf and Eddy 1914). The first construction of an intercepting combined sewer in this country was in Boston in 1876 (Foil et al. 1993). The intercepted sewage was usually transported to a primitive treatment plant consisting of solids and floatables removal via screening and settling (Metcalf and Eddy 1914).

During this period there was considerable debate between proponents of separate systems and those who favored CSS. The appeal of the combined system was one of economics, especially in areas where rainfall intensity was high enough to regularly flush the sewers, greatly alleviating the need for regular cleaning (Metcalf and Eddy 1914). While engineers in England were strongly advocating separate systems as early as 1842, primarily for sanitation reasons, engineers in America were divided. An important engineering monograph of the time by Dr. Rudolph Hering is quoted in "Design of Sewers" by Metcalf and Eddy (1914):

> ***The advantages of the combined system over a separate one depend mainly on the following conditions: Where rain-water must be carried off underground from extensive districts, and when new sewers must be built for the purpose, it (combined sewers) will generally be cheaper. But more important is the fact that in closely built-up sections, the surface washings from light rains would carry an amount of decomposable matter into the rain-water sewers, which, when it lodges as the flow ceases, will cause a much greater storage of filth than in well-designed combined sewers which have a continuous flow and generally, also, appliances for flushing.***

Thus problems associated with settled solids (e.g., maintenance costs and odor problems) were a primary reason for the spread of combined sewers in this country at the turn of the century.

Separate systems were advocated for areas with potable water concerns. Perhaps the "link" between wastewater and stormwater with drinking water in the urban water cycle was more evident under early 20th century conditions, when pumping costs were too great to accept the volume of combined sewage, and when rainwater did not require removal (Metcalf and Eddy 1914). One of the first separate systems designed in this country was in Memphis, TN following a yellow fever outbreak in 1873 when more than 2,000 persons died. Unfortunately, this system was apparently designed without regard to English experience and had significant design problems associated with it (Metcalf and Eddy 1914, Foil et al. 1993).

Separate sewer systems became more widely accepted as receiving water quality decreased and potable water supplies were threatened. They were designed primarily for newer urban areas, but later were also used as a means of doing away with combined systems. Separate systems, consisting of sanitary and storm sewers, remain the norm in the U.S.

However, NPS pollution has become more of a concern for urban areas (as well as in rural agricultural areas), separate untreated stormwater conveyance is now being questioned as an acceptable design practice. For example, sewer separation, a common mitigative action for areas with severe CSO problems, has been shown in some areas to be an infeasible solution for reducing water quality impacts. In Cincinnati, OH separation of the combined system was evaluated as a design alternative and shown to be an ineffective means of controlling the total solids load to the receiving water due to the polluted stormwater runoff from the untreated separate storm sewers (Zukovs et al. 1996). Conversely, separation has been an effective CSO abatement alternative in other urban areas (e.g., Minneapolis, MN). These cases indicate the site specificity of runoff, specifically with regard to land-use density and local rainfall characteristics. Clearly, a new look at some of these age old urban water management problems is in order.

Skokie, IL offers one example of a “new look.” Faced with a massive basement flooding problem caused by combined sewer surcharging, Skokie found traditional sewer separation to be technically feasible but unacceptably costly. Accordingly, controlled on and below street storage of stormwater was found to be a cost-effective (one-third the cost of separation) solution. Flow and storage control is achieved with a system of street berms and flow regulators. The premise of this retrofit system, which is almost completely implemented throughout the 8.6 square mile community, is that “out of control” stormwater is the root cause of combined sewer problems. As a side benefit, the Skokie system includes numerous pollutant-trapping sumps (Walesh and Carr 1998).

Problems Commonly Associated with Present Day Collection Systems

As described above, some collection systems in use today in the U.S. represent over 100 years of infrastructure investment. During that period the technical knowledge of the nature of wastewater has increased and the public expectation of the performance and purpose of collection systems has changed. What was considered state-of-the-art pollution control in 1898 is no longer acceptable. The societal goals which the engineer attempts to satisfy with a combination of technical feasibility and judgment have undergone drastic changes in the last 30 years (Harremoes 1997). Present day collection systems; many of which were designed and constructed in older periods when performance expectations and technical knowledge were less advanced than today, now must perform to today’s elevated standards. At the same time, sprawling urban growth has strained infrastructure in many areas, exacerbated by poor cradle-to-grave project management (Harremoes 1997). Designers of new collection systems must recognize and address the problems of past designs.

The current status of collection system infrastructure in the U.S. represents a combination of combined, sanitary and separate storm sewers. These collection systems vary in age from over 100 years old to brand new. While general design practices in the U.S. today are not drastically different than 30 years ago, current innovative research in Europe and Japan suggest that broad societal goals such as “sustainability” are not being achieved by current design practices in the U.S. Old combined sewers discharge raw sewage to receiving waters. I/I is a costly and wasteful problem associated with sewers. Sanitary sewer overflows (SSOs) discharge raw sewage from failed or under-designed separate systems. NPS pollution associated with urban areas is discharged from separate storm sewers. Proper transport of solids in sewers is still a misunderstood phenomenon, causing significant operational problems such as clogging, overflows, and surcharging.

This section provides an overview of the problems commonly associated with collection system infrastructure currently in use in the U.S. Designers of new collection systems must recognize these problems and address them with modern tools. Unsustainable design practices must not be allowed to be perpetuated in the field of urban water management. The useful life of the infrastructure is too long to simply design big systems to compensate for uncertainty. Following this section are sections describing innovative technologies being investigated and ways they might be used in the 21st century.

Combined Sewer Systems

CSS now constitute one of the remaining large-scale urban pollution sources in many older parts of major cities (Moffa 1990). In large urban areas, raw sewage, combined with stormwater runoff, regularly discharges to receiving waters during wet-weather. Water quality problems arise from NPS pollution in the stormwater portion of the discharge mixing with the sanitary wastes associated with the combined sewer. Low dissolved oxygen, high nutrient loads, fecal matter, pathogens, objectionable floatable material, toxins, and solids all are found in abundance in combined sewage (Moffa 1990). This mixture has led to some of the more difficult control problems in urban water management. However, CSS problems of today are the result of technology dating back to 1900 and earlier.

The traditional way to control CSO is to first maximize the efficiency of the existing collection system. This may include an aggressive sewer cleaning policy to maximize conveyance and storage properties of the system, reducing the rate of stormwater inflow, a re-evaluation of control points (frequently resulting in raised overflow weirs to maximize in-line storage in a static sense), and alterations of the wastewater treatment plant's operating policy to better accommodate short-term wet-weather flows (Gross et al. 1994). These measures were instituted as requirements for CSO discharge permits in 1994 by the EPA. The "Nine Minimum Control (NMC) Requirements" are (USEPA 1995b):

1. Proper operation and regular maintenance programs for the sewer system and CSO points.
2. Maximum use of the collection system for storage.
3. Review and modification of pretreatment programs to assure CSO impacts are minimized.
4. Maximization of flow to the WWTP.
5. Prohibition of dry-weather CSO discharges.
6. Control of solids and floatables.
7. Pollution prevention programs that focus on contaminant reduction activities.
8. Public notification to ensure that the public receives adequate notification of CSO occurrences and impacts.
9. Monitoring to effectively characterize CSO impacts and the efficacy of CSO controls.

In creating these permit requirements, the EPA has mandated that all owners must, at a minimum, adhere to these relatively low cost management activities.

These measures were frequently not enough, and less passive means of controlling CSO have been adopted in many cities. Storage of combined sewage, both in-line and off-line, has been used in a number of locations to capture frequent storms and the “first flush” of large events. As the capacity in the collection system and treatment works increases when the runoff subsides, the stored combined sewage is returned to the system for treatment (Field 1990). While not completely doing away with CSO (e.g., overflows occur when storage capacity is exceeded), storage of combined sewage has been a cost effective CSO control method (Walker et al. 1994).

Sewer separation has also been used in the U.S. This means of CSO control is expensive and is usually reserved for limited areas where severe overflow effects are concentrated in dense urban areas. As stated earlier, this means of control is not always adequate if polluted stormwater is discharged untreated. Traditional approaches of CSO mitigation including storage and separation are well documented in the literature and for detailed information the reader is referred to Moffa, 1990; USEPA, 1991a, 1993, 1995a, 1995b, 1995c, 1996a; WEF 1994a.

Other CSO control technologies that have been used on a more limited basis include high-rate treatment in the form of vortex or “swirl” separation technology (frequently in combination with storage), disinfection (including chlorination and ultra violet), micro screening, receiving water storage methods (including the flow-balance or the “Swedish method” developed by Karl Dunker), wetland treatment, floatable traps, and operation optimization techniques such as real time control (Field 1990; WEF 1994a; Seiker and Verworn (Ed.) 1996). Included in the category of CSO control technologies used on a limited basis is the previously mentioned on and below street storage of stormwater with the purpose of eliminating surcharging (Loucks and Morgan 1995, Walesh and Carr 1998).

An interesting development regarding CSSs is that due to contaminated stormwater runoff from urban areas that require treatment, combined systems are now at least being considered for new urban areas in some parts of Europe. CSS may in fact discharge less pollutant load to receiving water than separate systems where stormwater is discharged untreated and sanitary wastewater is treated fully. In southern Germany, CSSs are being designed with state-of-the-art BMPs to reduce the volume of stormwater entering the system. With reduced stormwater input, the number and volume of overflows are reduced over a traditional “old-fashioned” CSO, thus only discharging CSO during large, infrequent events, when the receiving water is most likely to be at high flow conditions also. This concept is discussed in more detail in subsequent sections of this chapter titled “Innovative Collection System Design – The State of the Art” and “Future Directions: Collection Systems of the 21st Century.”

Inflow and Infiltration

Separate sanitary sewers serve a large portion of the sewered population in the U.S. These sewers are generally of smaller diameter than combined or storm sewers, and serve residential, commercial and industrial areas. While sanitary systems are not specifically designed to carry stormwater, per se, stormwater and groundwater do enter these systems. This is a common and complicated problem for sewer owners. So common, in fact, that the design of sanitary sewers must include I/I capacity, which may actually exceed pure sanitary flow rates (ASCE/WPCF 1982). The capacities of many collection systems are being exceeded well before the end of their design life, resulting in by-passes, overflows, surcharging and reduced treatment efficiency (Merril and Butler 1994).

Inflow

Inflow is defined as surface water entering the sewer via manholes, flooded sewer vents, leaky manholes, illicitly connected storm drains, basement drains (probably illicit in most areas) and by means other than groundwater. Inflow is usually the result of rain and/or snowmelt events.

Inflow, contrasted with infiltration, is generally easier to control by enforcement of regulations and through proper design of the sewer/surface water interface (ASCE/WPCF 1982). For example, in areas prone to nuisance flooding (such as development in riparian land), careful design of sewer vents and manholes can limit the amount of storm drainage entering the sanitary sewer. Water tight, elevated vents must be above a certain flood elevation, and solid manhole covers with half-depth pickholes will greatly reduce chances for surface waters leaking into the sewer (ASCE/WPCF 1982). Tests performed on manhole covers submerged in one inch of water indicate as much as 75 gpm leakage into the sewer depending on the number and size of holes through the cover (ASCE/WPCF 1982).

Enforcement of regulations restricting impervious areas from draining into the sewer will limit the amount of illicit stormwater entering the sewer (ASCE/WPCF 1982). A 1000 sq. ft. roof area may contribute nearly 11 gpm during a one inch/hour rain storm (ASCE/WPCF 1982). Foundation drains may also contribute drainage water that will quickly overload sanitary sewer systems. A careful examination of local conditions and regulations must be made before determining design inflow rates for a sanitary sewer. Frequently, regulations are difficult and expensive to enforce, and costly provisions may have to be made to eliminate illicit connections. As such, the costs of treating and pumping inflow must be weighed against the costs of enforcement and mitigative actions such as yard regrading, and expensive foundation drains. Every sanitary sewer will have some point at which the present value of mitigative actions is greater than the present value of future pumping and treatment costs. Inflow reduction beyond this point is not cost effective (ASCE/WPCF 1982).

Infiltration

Infiltration is defined as water that enters the sewer via groundwater. This usually occurs through leaky sewer pipe joints, manholes and service connections. Being a function of groundwater head above the sewer leak, infiltration can result from stormwater and/or snowmelt infiltrating into the ground and into the sewer. Thus a wet-weather event can trigger both inflow (usually a faster response to the system) and infiltration in the form of groundwater (ASCE/ WPCF 1982). During wet-weather, a fast increase in flow rate in the sewer is due to inflow and a delayed response during or following wet-weather is caused by storm-induced infiltration. This wet-weather-dependent I/I in a separate sanitary sewer may behave nearly as fast as a CSS and, in turn, trigger SSOs (Miles et al. 1996). Infiltration can also occur purely as a function of groundwater elevation, independent of wet-weather. During dry weather the night-time minimum flows found in the sewer are from pure infiltration. Infiltration is usually much more difficult and costly to control than inflow. A typical sanitary sewer with likely sources of I/I indicated is shown in Figure 6-1.

Current design standards usually require that a certain amount of infiltration be accounted for in the design of a gravity sanitary sewer. Infiltration rates are given in units of volume per time per mile of pipe, normalized by the diameter of the pipe. In the U.S., values are reported in units of gpd/inch diameter/mile (gpd/idm). The joint ASCE-WEF design guidance for gravity sewers gives general guidelines for the volume of infiltration that should be used in capacity calculations for the a sewer at the end of its design life. Variations of local guidelines in the U.S. are presented in Table 6-1.

Table 6-1. Variations of infiltration allowances among cities (ASCE/WPCF 1982).

Cities Reporting		Allowance (gpd/idm)
Number	%	
4	3.1	1500
4	3.1	1000
1	0.8	800
2	1.6	700
1	0.8	600
63	49.2	500
11	8.6	450 to 300
16	12.5	250 to 150
21	16.4	100
5	3.9	50
Total = 128	Total = 100.0	Weighted Average = 422

Note: gpd/idm x 0.000925 = m^3/day/cm diam/km

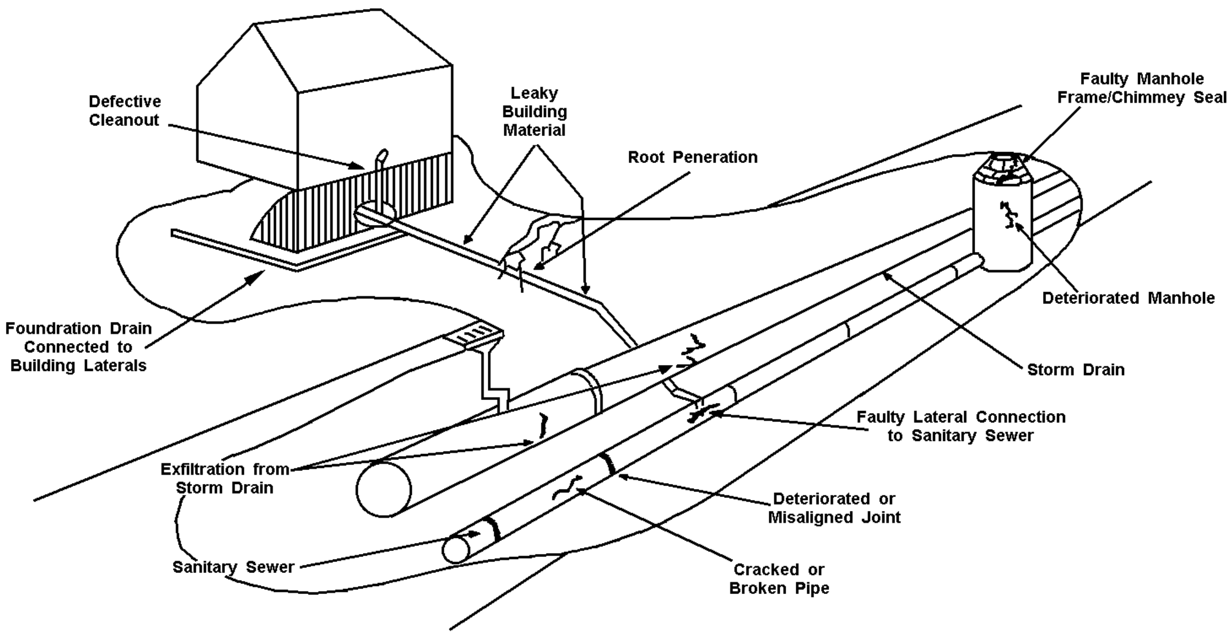


Figure 6-1. Typical entry points of inflow and infiltration (USEPA 1991a).

Inflow/Infiltration Analysis and Design Challenges

In existing sewers, the relative amount of I/I may be dramatic. Relative I/I contributions on an annual and monthly scale, respectively, are shown in Figures 6-2 and 6-3. The effect of groundwater elevation is evident in the annual analysis shown in Figure 6-2, where infiltration increases with groundwater. Inflow, on the other hand, tends to be a function of rainfall intensity, as seen in Figure 6-3.

A comparison was made of typical wastewater inputs versus the infiltration rates shown in Table 6-1 for an eight inch sanitary sewer. Typical wastewater flows were calculated for three population densities using 60 gpcd (DeOreo et al. 1996). Lateral spacing was assumed to be 50 ft. (high density), 100 ft. (medium density), and 150 ft. (low density). Each lateral was assumed to receive waste flows from four persons, thereby discharging 240 gpd. The results are shown in Figure 6-4. The conclusion from this theoretical comparison based on reasonable values is that typical infiltration rates allowed in the U.S. are a significant portion of the total wastewater flow.

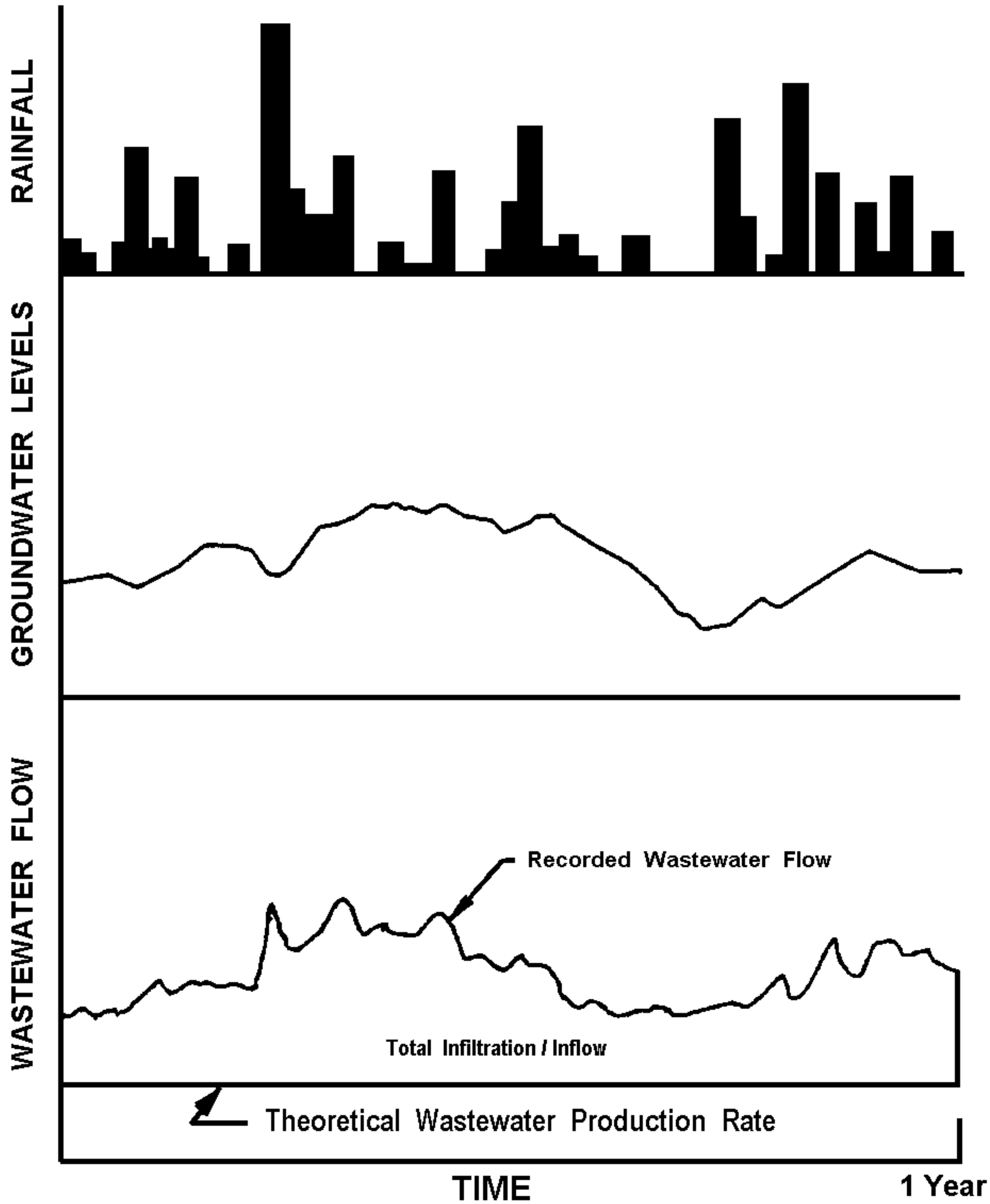


Figure 6-2. Annual contribution of I/I (USEPA 1991).

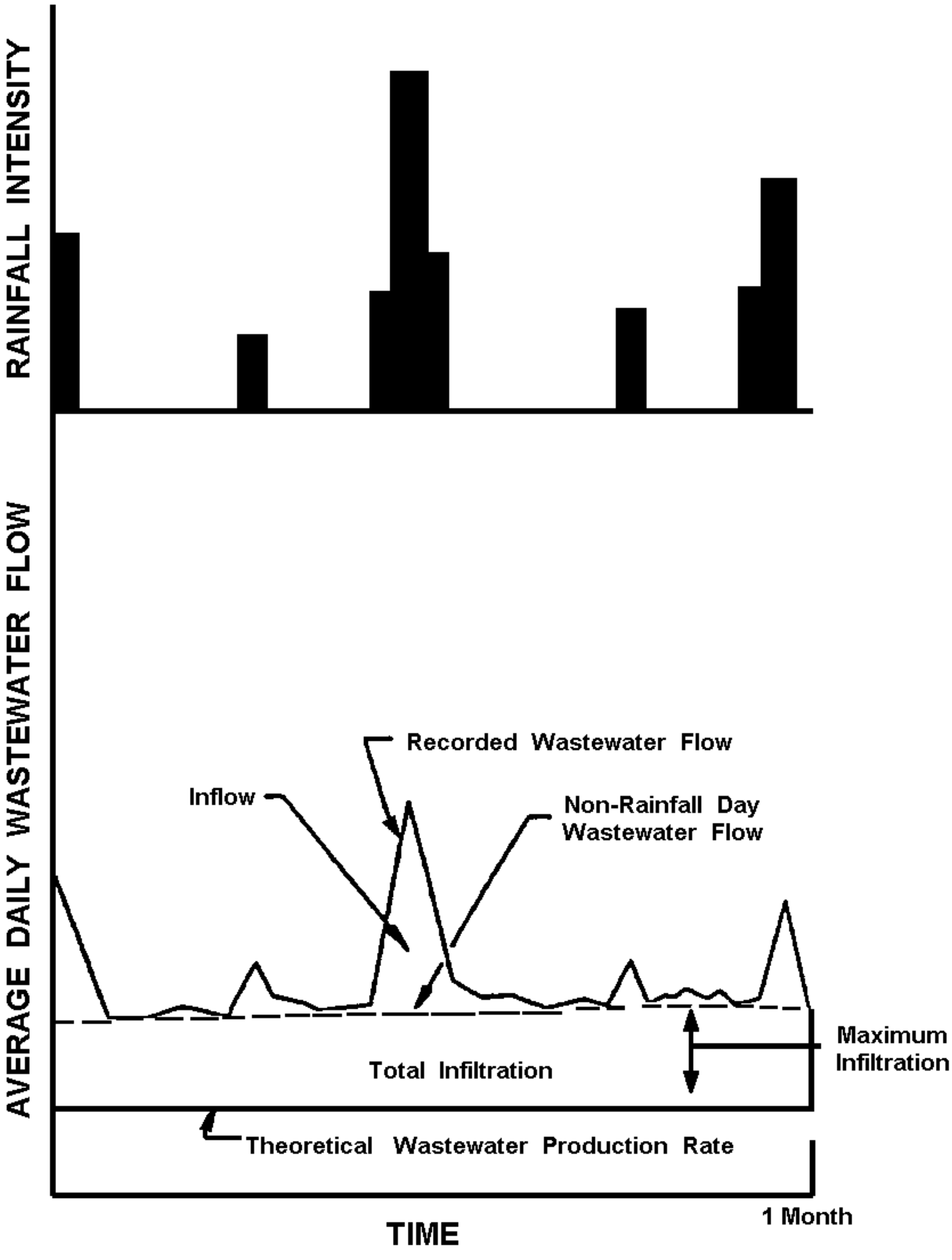


Figure 6-3. Monthly contribution of I/I (USEPA 1991a).

From Table 6-1, 50% of U.S. cities allow 500 gpd/idm or more. Table 6-2 shows the per capita I/I contribution for the three population densities for 500 gpd/idm. The results emphasize that infiltration is a significant portion of the wastestream, even using “moderate” rates such as 500 gpd/idm for an eight inch pipe.

Another comparison was made by using design values based on tributary area. Pre-1960s sewers were designed for 2,000 to 4,000 gal/acre/day I/I. Current design practice is 1,000 gal/acre/day. By comparison, per capita waste flow before 1960 was assumed to be 200 to 400 gal/capita/day, and the modern design value is 100 gal/capita/day (Heaney et al. 1997). The conclusion is that collection systems are designed for two to 10 times the dry-weather flow (Heaney et al. 1997). Therefore most of the sewer capacity presently “in the ground” is there to accommodate I/I (Heaney et al. 1997).

Table 6-2. Comparison of average daily wastewater and infiltration for one mile of 8 inch sanitary sewer based on 500 gpd/idm.

Population Density	Lateral Spacing (ft)	Population (four persons per lateral)	Per Capita Wastewater (gpd)	Total Wastewater (gpd)	Infil. (gpd)	Total (gpd)	Infil. (%)	Per Capita Infil. (gpd)
Low	150	141	60	8,460	4,000	12,460	32%	28
Medium	100	211	60	12,660	4,000	16,660	24%	19
High	50	422	60	25,320	4,000	29,320	14%	9.5

A review of 10 case studies in USEPA (1990) indicates that peak waste flows ranged from 3.5 to 20 times the average dry-weather flow (DWF). System surcharges would typically occur as the ratio reached 1:4 or 1:5 (USEPA 1990). Petroff (1996) estimated that I/I accounts for almost one half of the average flows to WWTPs in the United States. Houston, TX, measures peaking factors of 1:30 with maximum ratios reaching 1:50 (Jeng et al. 1996).

An example of the problems associated with reporting extraneous flows is found in a survey of 102 municipal wastewater management agencies from across the U.S. The survey was conducted by the Association of Metropolitan Sewerage Agencies (AMSA) and reported in AMSA (1996). The distribution of per capita wastewater flows and I/I from this survey is shown in Figure 6-5. The average per capita wastewater flow is 87.4 gpcd and average annual I/I is 37.4 gpcd (AMSA 1996).

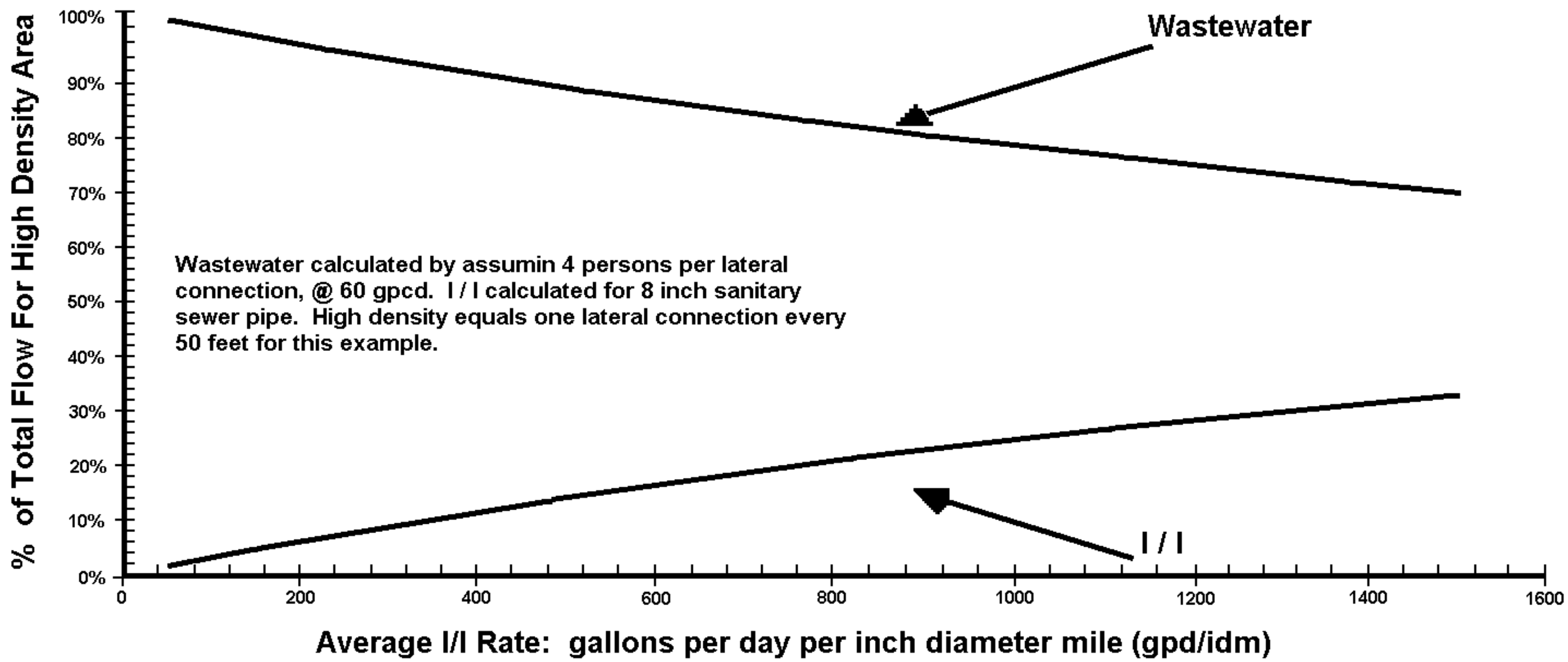


Figure 6-4a. Comparison of infiltration flow rates and residential flow rates for a one mile long, eight inch sanitary sewer (high population density).

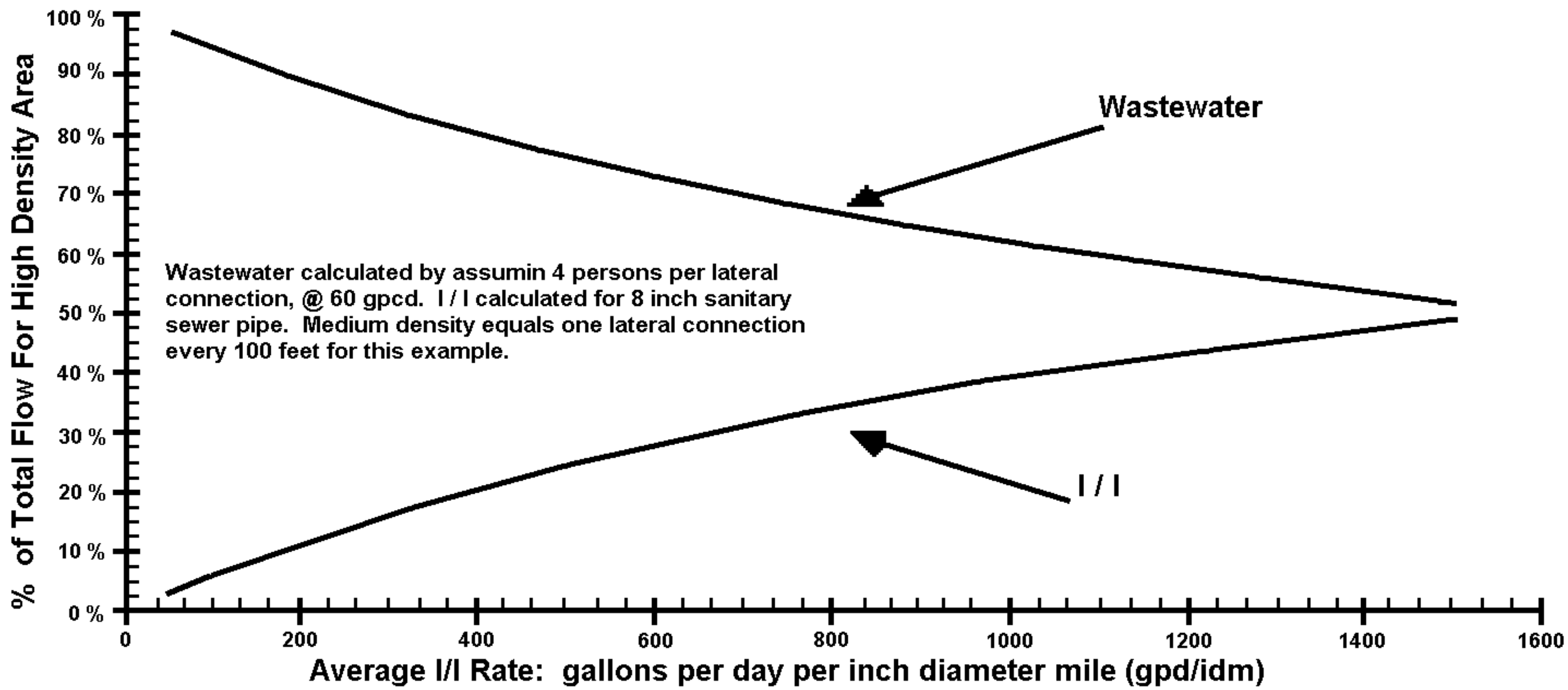


Figure 6-4b. Comparison of infiltration flow rates and residential flow rates for a one mile long, eight inch sanitary sewer (medium population density).

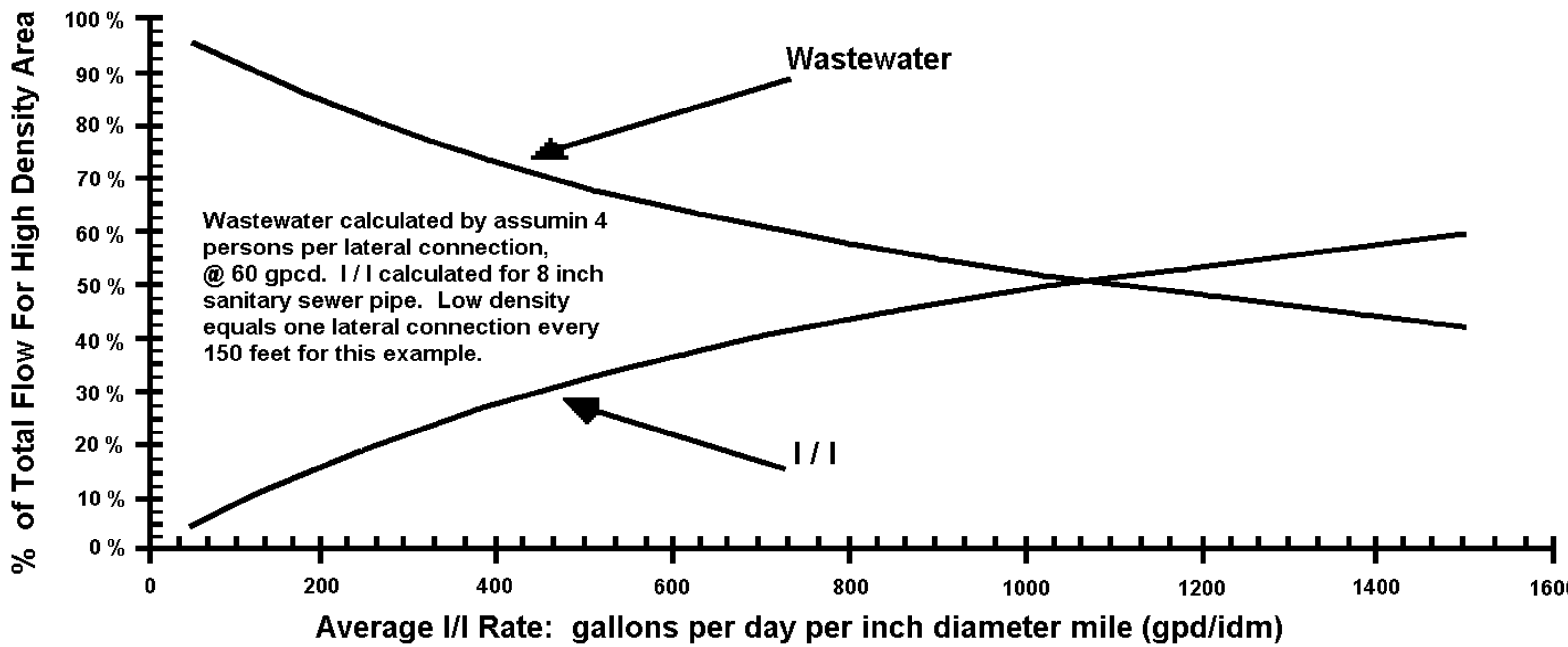


Figure 6-4c. Comparison of infiltration flow rates and residential flow rates for a one mile long, eight inch sanitary sewer (low population density).

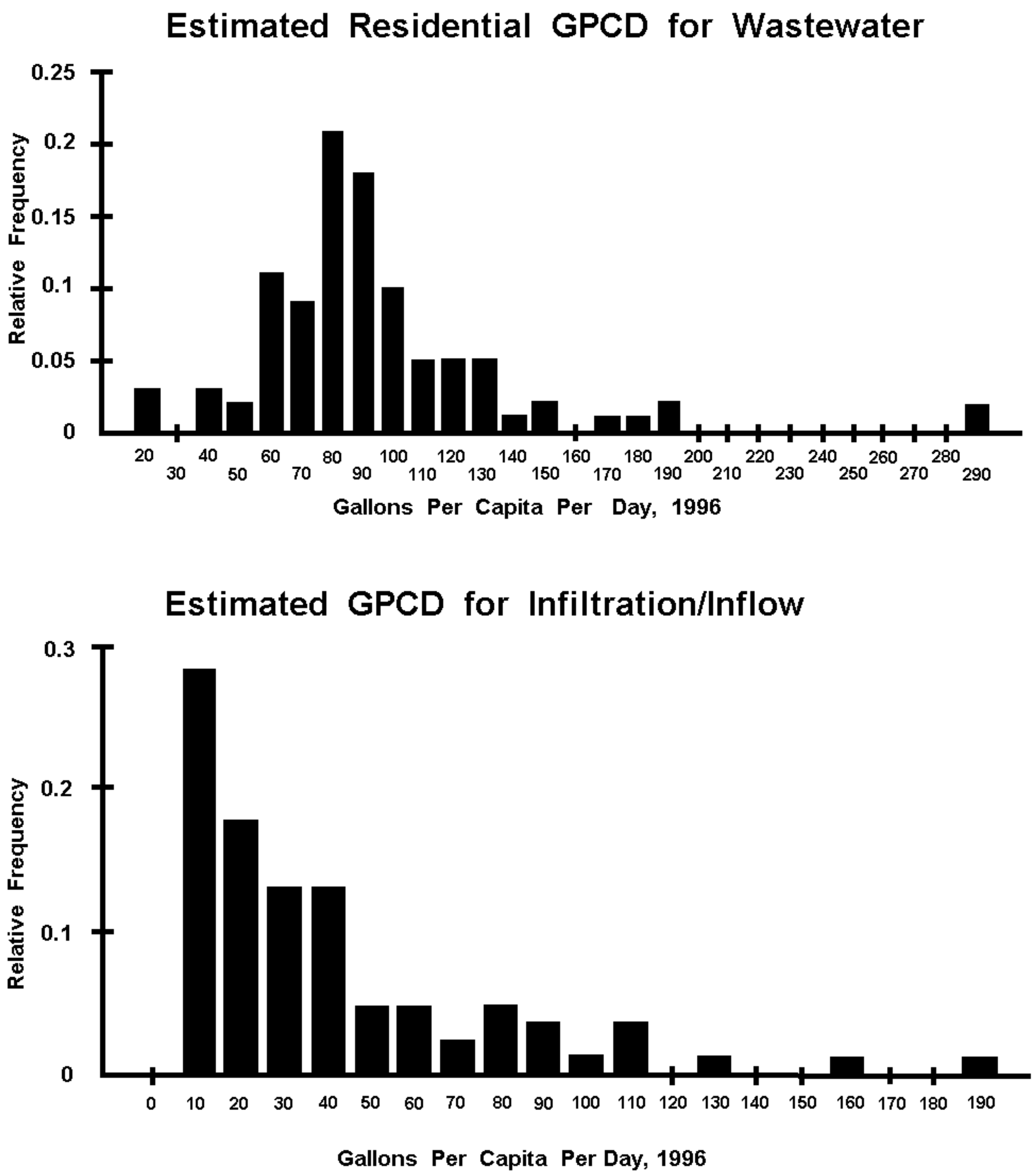


Figure 6-5. Histogram of average annual residential wastewater and I/I rates on a per capita basis from 102 U.S. cities (AMSA 1996).

Actual residential wastewater use, however, was found by DeOreo et al. (1996) to be 60 gpcd with little variance. Also, the I/I flow values reported in the AMSA survey are lower than reported I/I values on a national basis reported from other sources (e.g., Petroff 1996). It is likely that I/I values are under-reported in the AMSA survey, the difference being in inflated residential rates. For instance, the difference of reported residential and actual wastewater is 27.4 gpcd (87.4 - 60 gpcd). Added to the average reported I/I value, 37.4 gpcd, the result is an average annual I/I flow of 64.8 gpcd from across the nation. This value is in closer agreement with other sources, and highlights the fact that I/I can be a nebulous, imprecise quantity to estimate.

Methods of I/I detection are usually part of a complete Sewer System Evaluation Survey (SSES), which may include flow monitoring, pipe and manhole inspection, smoke testing, dye trace testing, and remote video surveys to isolate areas of high I/I (Rudolph 1995). These methods provide data that help locate areas with deteriorated sewers. Further analysis will identify areas contributing the most volume per sewer length and, therefore, the most likely areas for rehabilitation. Various methods are available for rehabilitation, including sewer lining, sealing, and reconstruction. Traditional approaches to I/I rehabilitation may be found in USEPA (1991a), ASCE/WPCF (1982), WEF/ASCE (1994), Read and Vickridge (Ed.) (1997).

Fixing an I/I problem can be an expensive rehabilitation project. It is only cost effective when the present value of the future costs of pumping and treating the I/I exceed the rehabilitation costs over the design life of the sewer including rehabilitation (WPCF 1982). Some older sanitary sewers may in fact have been designed to accept infiltration in order to dewater areas that may suffer damages from a high groundwater table. Other failing sewers may be providing the same function, though not originally designed to function this way. The added costs of damages resulting from high groundwater tables must be accounted for in an I/I evaluation. I/I rehabilitation policy must address this potential problem, as residents are likely to blame the I/I rehabilitation as “causing” groundwater flooding if they have been accustomed to this benefit for some time.

One problem associated with estimating and measuring I/I in existing sewers is the lumping or combining of inflow and infiltration. While both are sources of extraneous flow, they originate from different sources, tend to impact the system on greatly different time scales, and have different remedial measures. A likely reason that inflow and infiltration are combined together is the typical downstream “lumped” flow measurement at the WWTP headworks. For cost purposes, because inflow and infiltration are both extraneous to the waste stream, I/I are treated together.

This combining has led to confusion in reporting measured values in terms of average or peak flows for design or costing calculations. For pumping and treatment costs, average annual volumes are used for power and equipment cost estimation. In this case, reporting I/I together is correct. For other purposes, flow rates are important. Lack of frequency and duration of peak flows has exacerbated the uncertainty associated with extraneous flows. For example, the values in Table 6-1 were taken directly from a modern design guidance. While the figures in Table 6-1 only represent infiltration, there is little or no discussion as to whether these flows are an average flow over a year, a season, or day. If these are taken to be design allowances for additions to existing sewers, what is the return period of the rates given? This has design ramifications for the expected performance of the system at the end of its design life and the frequency of failure (e.g., surcharging and overflows).

Estimation of flow for wastewater design purposes has historically been more of an art than a science. While recent research has shown little variability in residential wastewater flows (DeOreo et al. 1996), designers have had to estimate peak and average I/I flows such as presented in Table 6-1 and 6-2 and in Figures 6-2 and 6-3. For new sewer design, inflow into the system can be expected to be insignificant if a surface drainage system is designed properly and if illicit connections are reduced by enforcement of local regulations (ASCE/WPCF 1982, Tchobanoglous 1981).

Expected infiltration rates at the end of the project life are uncertain and, therefore, must be estimated by the designing engineer. The uncertainty is due to site specitivity of soil and groundwater conditions and uncertainty of the expected future performance of modern construction techniques. For estimating peak infiltration rates, old systems range from 10 m^3/ha-d for 5,000 ha service area to 48 m^3/ha-d for 10 ha service area, and new systems range from 3 m^3/ha-d for 5,000 ha service area to 14 m^3/ha-d for 40 ha service area (Tchobanoglous 1981). The assumption is that performance has increased due to improved construction. While this is very likely true, to truly estimate life cycle costs the designer needs additional information on the frequency and duration of infiltration rates. The absence of a definition for “peak” in terms of time period (e.g., hour, day, season) and frequency (e.g., equaled or exceeded once every ten years) is very important for estimating performance. This information can only come from long-term, continuous measurement. Likewise, “average” infiltration rates for new sewers, without a definition of the return period or the duration of the average range from 2 m^3/ha-d for 5,000 ha service area to 9 m^3/ha-d for 40 ha service area (Tchobanoglous 1981). In the future, after a period of time when actual extraneous flows have been continuously measured for a variety of systems and in a variety of areas, flow/duration information will be available to reduce the uncertainty in extraneous flow estimation. Until that time, collection systems owners will continue to operate under a large cloud of uncertainty.

Reducing the amount of I/I in new sewers for the entire life of the collection system to near zero is imperative. This is critical from a variety of viewpoints. From a pure cost standpoint, the costs of treating I/I over a long period of time are large. From a design standpoint, the expected I/I from current systems near the end of their useful life may exceed sanitary flow and “drive” the design. In other words, if I/I can not be reduced to near zero, the designer must increase sewer design capacity to account for it. The sewer owner pays for a larger system than is required by societal demand, and then must pay to treat the I/I over the entire project life. Clearly this is not cost efficient or sustainable if the system can be constructed and designed from the outset as “tight”. Generally, the added costs of I/I-proofing the sewer will be far less during original construction than being forced to pay for expensive rehabilitation projects well into the lifespan of the system. As an integral part of overall urban water management, I/I control for new collection systems should be considered a major design objective.

In most cases, excessive I/I can be traced to poor construction techniques and materials and/or poor enforcement of policies regarding illicit connections. Current bidding practice is designed to minimize initial costs on the part of municipalities. However, the goal should be to minimize life-cycle costs given a certain level of performance over the entire project life. For the sewer owner of the 21st century (who may not be a public entity), measures must be taken to ensure that the construction and design contractors have a vested interest in the acceptable long-term performance of the collection system.

Sanitary Sewer Overflows

When the capacity of a sanitary sewer is exceeded, untreated sewage may discharge to the environment. SSO may be due to excessive I/I, from an under-designed (or over-developed) area releasing more sanitary flow than the system was designed for, from a sewer blockage, or from a malfunctioning pump station. The distribution of SSO causes from a sample of six communities is shown in Figure 6-6. An SSO can occur at the downstream end of a gravity sewer near the headworks of a WWTP or at relief points upstream in the system. These relief points may have been designed into the system, or retrofitted to alleviate a problem, or unexpected surcharging through manholes, basements or sewer vents. SSO causes from two case studies, in Fayetteville, AR, and Miami, FL., are shown in Tables 6-3 and 6-4. These data show that I/I is a significant cause of SSO, again reinforcing the importance of the need for data measurement discussed in the previous section.

SSOs are undesirable under any circumstance because they result in relatively high concentrations of raw sewage flowing directly to surface waters. Wet-weather SSOs may behave in a fashion similar to CSOs in extreme cases, though rehabilitation of the system is different. Instead of treating overflow (as is often the case of CSOs where the CSS provides primary drainage), wet-weather SSOs are more typically treated by attempting to remove wet-weather sources or removing hydraulic-capacity bottlenecks. Dry-weather SSOs are especially unwanted because the receiving water may not be running as high as during wet-weather, thus triggering more severe water quality degradation. Heaney et al. (1997) address a more detailed discussion of the relationship between wet-weather triggered overflows and receiving water assimilative capacity.

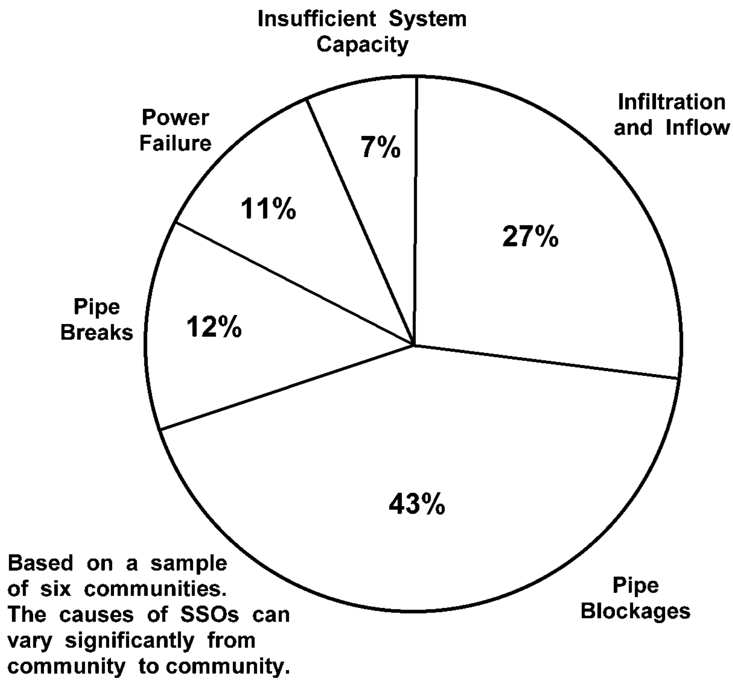


Figure 6-6. Estimated occurrence of SSO by cause (USEPA 1996b).

Table 6-3. Causes of SSOs in Fayetteville, AR (Jurgens and Kelso 1996).

Cause of SSO	1991 (%)	1992 (%)
I/I	39	36
Roots	19	24
Grease	25	13
Roots/grease	6	7
Other	11	20
Total (%)	100	100
Number of SSOs	161	123

Table 6-4. Causes of SSOs in Miami, FL (Clemente and Cardozo 1996).

Cause of SSO	Percent of Total
Pipe breaks (deterioration and accidental)	36
Pump station failures	30
Insufficient capacity due to wet-weather	19
Pipe blockages	15
Total	100

For new collection systems, the reasons for SSOs need to be thoroughly understood. Relief points for excessive flow during wet-weather events in sanitary sewers should not be a design concern if I/I is truly minimized. Likewise, if land use management plans are properly coordinated with system design and operation, then sewer capacity should not cause SSOs. However, surcharging due to clogging may occur even under the most rigorous of maintenance programs. Therefore, a pipe failure analysis should be conducted in the design phase to understand the reliability of the system. Relief points near the headworks of the WWTP should also be part of the design, to protect the treatment plant from possible excessive flows from unexpected sources. For example, a failure scenario could include a water main break that floods the sewer, or extreme surface water flooding that enters via non-illicit means, such as external sewer. In general, an integrated urban water management program of the future will have a minimum of SSOs, but collection system owners and regulators should at some point in the project life expect that some form of discharge due to surcharging will occur.

Separate Stormwater Collection Systems and Non-Point Sources

Separate storm sewers of one form or another can be found in virtually every municipality in the U.S. They are typically designed to collect stormwater from urban/suburban areas to prevent nuisance flooding (e.g., usually storms with return frequencies less than 10 years). This "level of protection" from flooding replaces an economic efficiency analysis that would ideally be performed on the basis of the worth of the potential damages resultant from flooding (ASCE/WEF 1993). The selection of return period is related to the exceedance probability of the design storm and not the reliability (or probability of failure) of the drainage system (ASCE/WEF 1993). Typical different levels of protection depending on the land use of the service area are presented in Table 6-5.

Table 6-5. Typical design storm frequencies (ASCE 1993).

Land Use	Design Storm Return Period (years)
Minor Drainage Systems	
Residential	2-5
High value commercial	2-10
Airports (terminals, roads, aprons)	2-10
High value downtown business areas	5-10
Major Drainage System Elements	< 100

A more thorough analysis of the expected performance of a drainage system would include a continuous mathematical simulation of the response of the system over an extended period of time using measured rainfall in the service area. This analysis would provide a more accurate estimation of the expected return period at which the capacity of the drainage system would be exceeded and the magnitude of the exceedances. This information may be used in conjunction with property values to estimate the distribution of expected damages that result from system exceedance thus providing a more rational basis of design (USACE 1994). In addition, the quality of the discharged stormwater may be mathematically simulated, which would provide information that could be used for receiving water management decisions. A detailed account of the benefits of continuous storm drainage accounting is provided in Heaney and Wright (1997).

Typical elements of a stormwater system include curbs, gutters, catchbasins, subsurface conveyance to a receiving water, sometimes first passing through a passive treatment facility such as a dry detention pond, a wet pond, and/or through a constructed wetland (ASCE/WEF 1993). This typical system may have open channels or swales instead of catchbasins and pipes.

Separate storm sewers may transport various forms of diffuse or NPS pollution to the receiving water. The amount and type of contaminant transported is heavily dependent on the land use of the tributary area, the rainfall/snowmelt characteristics of the area, and the type of storm sewer. Recent studies have shown a relationship between the impervious tributary area and receiving water quality. While the volume and time to peak of storm hydrographs have long been known to be adversely impacted by imperviousness, the water quality degradation aspects of imperviousness are still not completely understood.

Solids and Their Effect on Sewer Design and Operation

The fundamentals of modern sewer design haven't changed in many respects since the beginning of the century. Review of "Design of Sewers" by Metcalf and Eddy (1914), indicates that the fundamentals of minimum and maximum velocities, grade, flow rate prediction, and solids transport were in place at the turn of the century after hundreds of years of trial and error designs dating back to ancient civilizations. Modern design has significantly refined the information used in design, but the basic engineering criteria have remained, much to the credit of early sanitary engineers.

The purpose of sewer collection systems has always been to safely transport unwanted water and solids. Historically, sewer design has focused primarily on the volume and flow rate of the fluid, and has assumed solids will be carried with the fluid if certain "rules-of-thumb" regarding velocity are followed. This imprecise method of designing for solids transport has been a costly and significant source of maintenance needs over the years in the U.S. and elsewhere.

Recent research conducted in Europe (Ashley (Ed.) 1996) has focused on the age-old question of transport of solids in sewers. The flow rate, velocity and size of pipe are all important in determining the amount and size distribution of solids a particular sewer will carry. Therefore, along with flow rate, the solids transport question is one of the most fundamental questions that must be addressed when calculating costs. It is a vexing question, because solids transport is a function of flow rate, velocity, pipe size, pipe material, gradient, solids concentration, size distribution of the solids, and type of solid (e.g., colloidal or non-colloidal, and grit). Also important is the question of solids transformation in the collection system. Fundamental research conducted in Europe has shed some light on this issue (Ashley (Ed.) 1996, Sieker and Verworn (Ed.) 1996, Ackers et al. 1996).

A historic reference to a minimum design velocity is found in Metcalf and Eddy (1914), where an early sewer design in London is cited as using a value of 2.2 fps to avoid unwanted deposition in sewers. Other early work on minimum grades for various pipe sizes was done by Col. Julius W. Adams in designing the Brooklyn sewers in 1857-59 (Metcalf and Eddy 1914). Col. Adams' recommended sewer grades are shown in Table 6-6, and compared with modern values found in Gravity Sanitary Sewer Design and Construction (ASCE/WPCF 1982). These early designers recognized that the minimum mean velocity to avoid deposition was dependent on the pipe diameter.

However, in the 1994 WEFTEC proceeding "Collection Systems: Residuals and Biosolids Management", a paper entitled "Two feet per second ain't even close" by P. L. Schafer discusses the problems associated with deposition in large diameter sewers due to using a "rule-of-thumb" design value of two fps (Schafer 1994). Modern design guidelines still state: "Accepted standards dictate that the minimum design velocity should not be less than 0.60 m/sec (2 fps) or generally greater than 3.5 m/sec (10 fps) at peak flow." (ASCE/WPCF 1982). One problem with this recommendation is the lack of peak flow definition. Should this be the seasonal, monthly, daily, or hourly peak flow? The frequency and duration of the flushing flow are critical to the proper performance of the sewer. Ideally, a settled sewer particle at the furthest end of the collection system will be re-entrained into the waste stream and carried to the WWTP. Clearly the minimum velocity design problem has not been resolved.

Sewers that exhibit sediment deposition are prone to a multitude of problems over time. Excess sedimentation promotes clogging, backwater and surcharging and may promote corrosion by producing hydrogen sulfide (Schafer 1994). Because sedimentation problems are more likely to occur in larger diameter sewers, such as trunk sewers, the associated costs of sewer failure may be substantially greater than in a smaller diameter pipe. In combined systems, the in-line storage that is taken up in a heavily sediment-laden trunk or interceptor sewer will tend to increase the volume and frequency of overflow events (Mark et al. 1996). In addition, the deposited sediments in combined systems represent a build up of pollutants. that may resuspend during wet-weather (Gent et al. 1996).

Table 6-6. Comparison of recommended minimum sewer grades and velocities over the years.

Source	Type of sewer and pipe diameter	Minimum Slope (ft/ft)	Minimum Velocity (fps)
Balzalgette, London, c. 1852 (1)	Large intercepting sewers – combined system		2.2
Roe, London, c. 1840 (1)	Large intercepting sewers – combined system	0.002	
New Jersey Board of Health, 1913 (1)	8" – Sanitary sewer (n = 0.013)	0.004	
"	12" – Sanitary Sewer (n = 0.013)	0.0022	
"	24" – Sanitary Sewer (n = 0.013)	0.0008	
Metcalf and Eddy, 1914 (2)	Combined systems		2.5
"	Sanitary systems		2.0
WPCF/WEF 1982 (3)	Sanitary systems		2.0
WEF/ASCE 1992 (4)	Storm sewers		2 - 3
Acker et al. 1996 (5)	150 mm (5.9 in)	0.0062	2.2
"	225 mm (8.85 in)	0.0043	2.36
"	300 mm (11.8 in)	0.0032	2.46
"	450 mm (17.7 in)	0.0024	2.59
"	600 mm (23.6 in)	0.0021	2.95
"	750 mm (29.5 in)	0.0022	3.48
"	1000 mm (39.3 in)	0.0025	4.43
"	1800 mm (70.8 in)	0.0028	6.66

Note:
1. Col. Julius W. Adams (c. 1859) in Metcalf and Eddy (1914)
2. Metcalf and Eddy (1914)
3. ASCE/WPCF (1982)
4. ASCE/WEF (1992)
5. Ackers et al. (1996)

The movement of solid material in flowing water is a complex phenomenon that depends on the nature of the solid particles, the nature of the flow, and the nonlinear interaction between the two. A solid particle undergoes acceleration from the force of gravity, from the average advective motion of the water, and from the local turbulent motions of the water. Particles may be suspended in the water column of the sewer, deposited along the bed of the sewer, or slowly move along the bedload of the sewer. Once deposited under low flow conditions, a particle may resuspend into the water column under high flow conditions. In addition, a particle may exhibit cohesive properties, adjoining with other particles both in suspension or in the bed after deposition. Sewer particles may be organic, with low specific gravity and break down both physically and biologically while in the sewer.

When considering sewer collection systems, the proper transport of solids is crucial to a correctly functioning system. There are distinct areas where deposition should be avoided, (e.g., the conduit network) and also areas where deposition is desired, (e.g., treatment works). The system should function under a wide range of hydraulic conditions and under a wide range of solid loadings. The solids may also vary widely in character, which may alter the performance of the sewer.

To avoid deposition, a common design method is to calculate the shear stress required to move the largest size of particle expected in the sewer under average or high flow conditions (Schafer 1994). This assumes that the frequency of the high flow is enough to avoid excessive deposition and the subsequent creation of a permanent bed layer. The critical shear stress of a particle is defined as the minimum boundary stress required to initiate motion (Schafer 1994). Chow (1959) indicates that shear stress is a function of the specific weight of water and the hydraulic radius and invert slope of the sewer. Various values of critical shear stress have been recommended, depending on the maximum size of particle found in the sewer. Values of critical shear stress recommended by various researchers are shown in Table 6-7.

Table 6-7. Recommended critical shear stress to move sewer deposits (Schafer 1994).

Recommended critical shear stress		Reference	Conditions
N/m^2	lb/ft^2		
4	0.08	Lynse 1969	Sanitary sewers
4	0.08	Paintal 1972	Sanitary sewers
1.5 to 2.0	0.03 to 0.04	Schultz 1960	German work
1 to 2	0.02 to 0.04	Yao 1974	Sanitary sewers with small grit size
3 to 4	0.06 to 0.08	Yao	Storm sewers
2.5	0.05	Nalluri 1992	Sand with weak cohesiveness
6 to 7	0.12 to 0.14	Nalluri 1992	Sand with high cohesiveness

Note: 1 N/m^2 equals 0.02064 lb/ft^2

Schafer (1994) recommends that the lower end of the shear stress range in Table 6-7 is adequate only for waste streams with small particle size and limited grit, and when flushing flows may be expected daily. The high end of the range is appropriate when the waste stream contains heavy grit and gravel, as is common in combined or storm sewers (Schafer 1994). Table 6-7 indicates that commonly used design values for the minimum flushing velocities in sewers are not adequate to scour grit from large sewers. Consider, for example, a 48 inch diameter sewer transporting a reasonable load of grit. Minimum velocities in the range of 4.0 fps are required to flush deposited grit, far greater than the 2.0 fps recommended in some design guidelines. However, European research shows that bed stress is the most important criterion, and a minimum bed shear stress of $2N/m^2$ is required to ensure sediment transport (Ashley and Verbanck, 1997).

Uncertainty in key design parameters is the source of unnecessary cost. If under-designed, operation and maintenance costs are likely to be high. If over-designed, additional unnecessary capital costs are incurred as are high maintenance costs due to solids deposition at low flows. Just as this was shown to be true in the discussion of I/I, so it is also true for designing sewers for solids transport.

However, in addition to the lack of high quality frequency/duration information regarding flows, the designer concerned with solids transportation must also contend with a physical process about which only the rudimentary nature is known. The relationship between the solids concentration, the distribution of settling velocities, and the dynamics of movement are not well understood for gravity pipe flow. Operational costs will be incurred if the frequency and duration of velocities are not enough to regularly cleanse the pipes. Deposition in uncleaned sewers will cause SSOs. Thus environmental costs are also incurred. If over-designed, the sewers will remain clean, however additional excavation and material costs will be incurred.

While attempts have been made to estimate costs of I/I and SSOs on a national basis, there are no cost estimations of improperly designed sewers. It is likely that these costs, if known, would dwarf those for I/I and SSOs. As is the case with I/I estimation, new systems that record and store operational data will be invaluable to improving design techniques for solids transport.

Predicting Pollutant Transport in Collection Systems

A problem associated with present day collection systems is that, given the current state of computer simulation technology and knowledge, simulating pollutant transport correctly through a complex collection system is very difficult. This is especially true if complex hydrodynamics and continuous simulation are required. Due to the complex nature of the governing hydrodynamic equations, coupled with sediment transport equations, continuous simulation of the response of a collection system is nearly impossible for realistic system configurations. However, the designer of new collection system should realize that this will likely not be true in the near future. Data retrieval via Supervisory Control And Data Acquisition (SCADA) systems should be considered a major system component in collection systems of the 21st century. Data acquisition will be imperative for real time control and advanced simulation/optimization and designers of new collection systems must recognize that the technology available at the end of the project life of the collection system will be far advanced from what is available today.

To properly simulate pollutant discharges from a sewer system, a model must have the ability to simulate the movement of solids in sewers (Gent et al. 1996). Research conducted in the UK has shown four types of sewer transport (Gent et al. 1996):

1. Suspended transport (occurs at or slightly slower than the flow rate).
2. A dissolved or very fine rate (occurs at the ambient flow rate).
3. A dense near-bed layer (occurs during periods of low flow).
4. A course bed load layer (occurs during periods of high flow or in steep sewers).

The near bed and bed layer are the primary pollutant transport mechanisms and are also the main sources of deposition (Gent et al. 1996). Current trends in mechanistic modeling of collection systems indicate that these transport mechanisms will be part of future mathematical models. It should be assumed that future collection systems will have extensive data collection systems and that computational capabilities will be advanced to the point of accurately simulating pollutant behavior in a pipe network.

Characteristics and Treatability of Solids in Collection Systems

When considering the transport and/or treatment of solids in sewers, the cumulative effect of gravity on the overall particle distribution must be measured. Sewer solids may occur in a wide range of specific gravities and an equally wide range of shapes. The settling characteristics of the entire distribution of solids must be known to properly establish solids behavior in pipes, pumping stations and treatment works. Due to the site specific nature of solids, local data on settling velocities are greatly preferred over literature values.

Several forms of measurement tests have been developed and Pisano (1996) provides a summary of the currently accepted techniques. All methods provide estimates of the distribution of settling velocities for a particular solids sample. However, the results are a function of the protocol used and, therefore, not absolute. Pisano (1996) shows an example plot of settling characteristics for various forms of sewer samples. Data show a wide range of “treatability,” that is, ability to settle as determined in laboratory tests. When considering design of new systems that include wet-weather treatment, a standardized measure of settling velocity distribution data will be needed.

Innovative Collection System Design - The State of the Art

Recent work in all aspects of sewer collection systems, from design and facilities-planning level research to construction and operation and maintenance, shows promise for greatly improved collection system performance for the next century. In addition, drastically new technologies are being considered which may greatly affect the future configuration of urban water management. Some innovators in the field are advancing ideas to replace water-intensive waste removal systems.

This section provides an overview of many aspects of sewer concepts. It is generally organized in terms of increasing innovation. In other words, the first examples remain closest to present day systems and the last innovations described deviate furthest from current design concepts. The reader is reminded that this section is an overview of innovative ideas in the field of waste management. Many of these ideas are only now being tested and inclusion in this guidance should not be misunderstood as a recommendation by the authors or the USEPA. References are provided for the interested reader to follow up on performance testing in the future. The section following this one attempts to provide these technologies in a future scenario-type context.

During the past decade, many changes in the understanding of global and local effects of urbanization, population growth, and land use have brought about a concern for future generations. This concern is manifested in a concept for future development called “sustainability” which is discussed in Chapter 3 of this report. While there are many interpretations of the concept of, engineers have attempted to bring the fundamental concepts to the practitioners and policy makers. In the field of urban water management, sustainability concepts are being used to critique current water management practices, and bring fresh ideas of waste management to decision makers. Henze et al. (Ed.) (1997) provide the most recent work in this area. Innovative collection system concepts attempt to reconcile problems discussed in the earlier section of this chapter titled “Problems Commonly Associated with Present Day Collection Systems.” While rethinking the whole concept of transporting urban wastes via underground water-driven sewers.

Recent literature in the area of sewer innovations were surveyed from WEF (1994a), WEF (1994b), WEF (1995a), WEF (1995b), WEF (1996), Sieker and Verworn (Ed.) 1996, Ashley (Ed.) 1996, Bally et al. (Ed.) (1996), Henze et al. (1997), USEPA (1991a), and USEPA (1991b). An especially important summary of vacuum, pressure and small diameter gravity sewers is presented in USEPA (1991b).

Current Innovative Technologies - Review of Case Studies

Data Management, SCADA, Real Time Control

Many fields, including that of urban water management, have barely been able to keep up with the rapid technological and computational advances of the past decade. This has been exacerbated in the U.S. by the relative longevity of civil infrastructure works and the amount of infrastructure already in place, the majority being constructed in the 20th century. As the end of the project life of many of these works is approaching, and as new urban areas are being contemplated for certain high-growth sections of the U.S., practitioners and researchers in the field of urban water management have a unique window of opportunity. Now is the time to take advantage of the latest in technological advances and to use the past two decades as a model to predict what the future may bring in terms of technology.

The information age has changed the way in which resources are managed. This fact will be more apparent in new collection systems and waste management of the 21st century. New systems will be operationally data intensive due to a higher level of control. The current level of control in WWTPs may be seen as extending into the collection system. The increase in data quality and quantity will have positive effects on simulation for design, simulation for operation and for real time control of the system. These innovations should decrease costs and environmental impacts and maximize utility of the system.

Seattle, WA was one of the first major municipal sewer owners in the U.S. to use real time measurements of the collection system in a control scheme (Gonwa et al. 1994, Vitasovic et al. 1994). Vitasovic et al. (1994) describes the use of Real Time Control (RTC) in Seattle for CSO control purposes. Vitasovic (1994) states the goal of the program succinctly:

> ***...the idea behind RTC of CSO's is fairly straightforward: the conveyance system is controlled in real time with the objective of maximizing the utilization of in-line storage available within the system. The cost of the control system is often a fraction of the cost required for alternatives that include construction of new storage facilities*.**

The Seattle experience highlights the need for some form of system simulation to test control procedures off-line and to provide a higher level of system knowledge on-line than from data measurement alone (Vitasovic et al., 1994). A SCADA system provides automation one level above manual process level control and interfaces data retrieval systems with a relational database (Vitasovic et al., 1994, Dent and Davis 1995). Under the SCADA level of control, operators usually manage the system from a centralized location using Man-Machine Interface (MMI) software, receiving data from the SCADA while maintaining a supervisory level of control over the system (Vitasovic et al., 1994, Dent and Davis 1995). A higher level of automation may be used if a computer controller is used to change system operation. This can include simple control algorithms such as if-then and set-level points, or may be as advanced as providing on-line non-linear optimization (e.g., genetic algorithms).

Other successful applications of RTC in the U.S. include Lima, OH, Milwaukee, WI, and Cleveland, OH. Gonwa et al. (1994) provide a summary of the Milwaukee upgrade of an existing RTC. One new feature of the upgrade was additional control applied to the headwork's of the WWTP.

The hydraulic grade line of the Milwaukee system modified by the RTC upstream of the

WWTP resulted 1,5000,000m^3 inline storage volume during peak storm diversions to ISS after interceptor storage is maximized. In other words, the RTC provides control of the system to maximize pipe storage before diverting to the Inline Storage System. RTC is used in combination with storage facilities to minimize overflows.

Most applications of RTC, SCADA, automated system optimization and other advanced data management techniques are currently used in collection systems designed before the computer/information-age revolution. For new collection system designs, it is imperative that designers understand the physical/structural requirements of long-term high-quality data measurement. Successful designs will have adaptivity “built-in”. The ability to change operational procedures as technological advances become available will greatly extend the useful life of future collection systems. In other words, future collection systems will have many critical “high information points” that, used in conjunction with control and simulation, will facilitate operating the system for optimal utilization. The tools used to accomplish this task will change during the project life of the system because of the longevity of infrastructure in contrast with the rapidity of computer technological advances. A successful design will anticipate these changes.

Sanitary Sewer Technology - Vacuum Sewers

Hassett (1995) provides a summary of current vacuum sewer technology. A typical vacuum sewer configuration is shown in Figure 6-7.

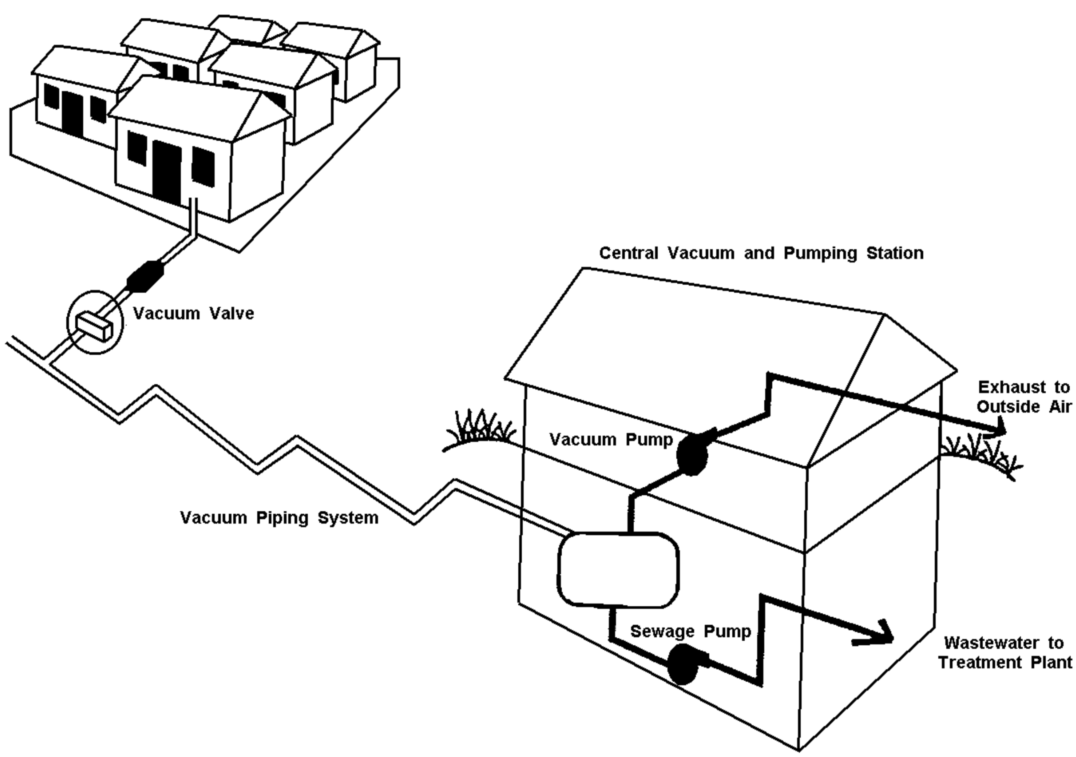


Figure 6-7. Typical vacuum sewer system schematic (Hassett 1995).

Vacuum sewers are typified by shallow pipelines that make them attractive for high-groundwater areas and for alignments that would require expensive rock excavation for gravity lines. Such systems are also useful in flat countries such as the Netherlands. Being completely sealed, vacuum lines also do not have any I/I - a remarkable benefit that begs the question: If vacuum sewer lines can be constructed water tight, why can't gravity lines? Vacuum systems do show promise, however, especially with recent advances in lifting capabilities. A recent installation in an Amtrak station in Chicago, IL used a valve configuration that achieved over 20 feet of vacuum lift (Hassett 1995). Another advantage of these systems is that vacuum toilets function with less than a third of water per flush than do modern low-flush toilets, using only 0.3 to 0.4 gallons per flush, compared with 1.5 to 6.0 gallons for toilets connected to gravity sewers.

Hassett (1995) provides a cost comparison for vacuum sewers for an actual project location in Virginia. The service area was assumed flat with a three foot depth-to-groundwater, an area of 750 acres (300 hectares), and approximately 750 residential units housing 3,000 people. The density was then varied to provide the construction cost information presented in Figure 6-8 and the operating costs shown in Table 6-8. Hassett (1995) notes that the operating costs of any of the system configurations is only 4 to 6% of the present value of the capital components and is, therefore, unlikely to be a decision factor. This observation may not be true in countries with higher energy costs.

Table 6-8. Annual operating costs of vacuum and gravity sewer systems as of 1995 (Hassett 1995).

Type of Sanitary Sewer System	Cost (1995 $U.S.)			
	Labor	Materials	Power	Total
Gravity (Dry)	26,000	3,000	4,000	33,000
Gravity (Wet)[1]	28,000	28,000	4,000	60,000
Modern Vacuum	42,000	10,000	8,000	60,000
High Lift Vacuum	34,000	3,000	8,000	45,000

(1) Wet means that the system includes lift stations and is below the water table.

A major advantage of these systems (along with pressure sewers) is their adaptability to monitoring and control. The use of pressure instead of gravity flow simplifies flow measurement. Control of these system is more exact than with gravity systems, thereby making them suited for overall system optimization by RTC.

Low Pressure Sewers

Another modern collection system technology that has been used in the U.S. is the low pressure sewer used in conjunction with a grinder pump (Farrell and Darrah 1994). These systems use a small grinder-pump typically installed at each residence. The grinder pump reduces solids to 1/4 to 1/2 inch maximum dimension (Farrell and Darrah

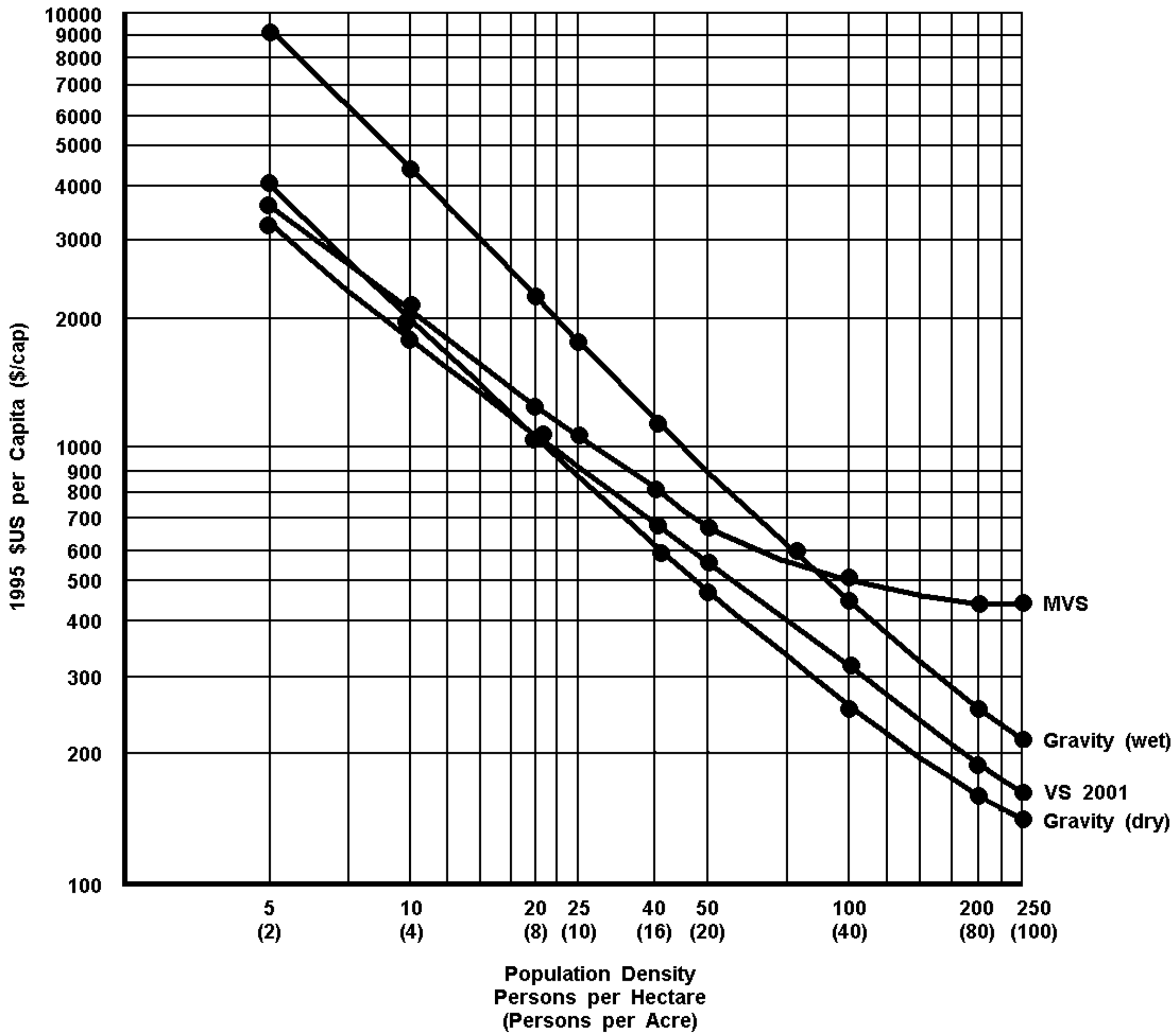


Figure 6-8. Per capita construction costs for different sanitary sewer systems at various population densities (Hassett 1995). (Note: MVS means modern vacuum system and VS 2001 represents 21st century vacuum system).

1994). Like vacuum systems, these low-pressure grinder systems feature water tight piping, thus virtually eliminating I/I. A full system in Washington County, MD went on-line in 1991. Water use, rainfall and wastewater flows were monitored and wastewater flows were found to be 110 to 130 gpcd, with no measurable increase during or following wet-weather events (Farrell and Darrah 1994).

A demonstration facility in Albany, NY was installed in 1972, where per capita flows were only 34 gpd. One purpose of this demonstration was to determine the effect of grinding solids on settleability. The conclusion was that there was no effect on settleability and treatability as compared with solids transported via a traditional gravity sewer (Farrell and Darrah 1994). Other demonstrations found no significant differences in grease concentrations (Farrell and Darrah 1994). The LPS pipe was excavated after several years of service, and no significant build-ups of solids were noted in the pipes (Farrell and Darrah 1994).

LPS systems have over a 20 year track record. As with most new technologies, engineers were hesitant to specify these sewers despite smaller capital expenditures due to the lack of long-term experience (Farrell and Darrah 1994). The reliability and costs of operating and maintaining the pumps were a major impediment to widespread use. Reliability of LPS systems has increased dramatically since the first commercial installation at a marina in the Adirondack mountains in NY (Farrell and Darrah 1994). In the 1972 Albany demonstration project (which only lasted 13 months), the mean time between service calls (MTBSC) for pump maintenance was 0.9 years (Farrell and Darrah 1994). An LPS system installed in 1986 in Bloomingdale, GA. averaged 10.4 years between service calls (Farrell and Darrah 1994) over an eight year period. Pump operation and maintenance (O&M) costs and MTBSC for five LPS collection systems are shown in Table 6-9 (Farrell and Darrah 1994).

Table 6-9. Pump data and O&M costs for low pressure sewer systems (Farrell and Darrah 1994).

Location	Number of Pumps	Average Age (years)	Annual O&M ($/pump)	MTBSC (years)
Cuyler, NY	41	17	53.00	4.6
Fairfield Glade, TN	955	16	36.07	5.6
Pooler/Bloomingdale, GA	998	11	13.24	10.4
Pierce County, WA	900	9	51.00	7.9
Sharpsburg/Keedysville, MD	780	5	18.00	>20

As with vacuum systems, LPS systems are well suited for control and monitoring due to the use of pressure rather than gravity to drive the system. This may be a significant advantage over gravity system in the future for RTC applications.

Small Diameter Gravity Sewers

These systems consist of a system of interceptor tanks, usually located on the property served, a network of small diameter collector gravity sewers (USEPA 1991b). The interceptor tanks remove settleable solids and grease from the wastewater. Effluent from each tank is discharged to the collector sewer via gravity or by pumping (septic tank effluent pumping (STEP)) (USEPA 1991b). A typical system layout is shown in Figure 6-9.

This system has the advantage of not having to transport appreciable solids (USEPA 1991b). Cost savings therefore result from having a lower required velocity and from less cleaning costs. Also, peak flows are attenuated in the tank. Therefore, the average to peak flow rate from wastewater is far less than for a standard gravity sewer (USEPA 1991b).

Otherwise, these systems function much the same as traditional gravity sewers. They have been used in rural areas to replace existing septic tank discharge. They are also used in developing countries to share costs (Mara, 1996) where they have been known as settled sewerage. A problem associated with these sewers is I/I. The use of old septic tanks tends to increase the amount of rainfall induced infiltration (USEPA 1991b).

Black Water/Gray Water Separation Systems

A more drastic break with modern systems is that of water separation at the household level. This has been a relatively active research area in recent years because of its appeal from a water conservation standpoint. Water from faucets, showers, dishwashers and clothes washers drains to a separate on-site filtration device. The filtered as water is then typically used for outdoor irrigation. This may be especially advantageous in arid areas where on-site stormwater detention for outdoor use does not meet the evapotranspiration needs on an average annual basis.

Waste/Source Separation

Recent research in Europe has focused on the separation of household waste in a variety of ways (Henze et al. (Ed.) 1997). The goal of these systems is to promote nutrient recycling and limit entropy gain (a goal for sustainability) via dilution. Urine separation is perhaps the most radical departure, where urine is tanked on site and converted to fertilizer (Hanaeus et al. 1997). Human urine contains 70% of the phosphorus and 90% of the nitrogen found in wastewater from toilets (Hanaeus et al. 1997). This technology is still in the formulation phase and has only been tested on a limited basis. Research shows it may have applicability for certain waste management applications.

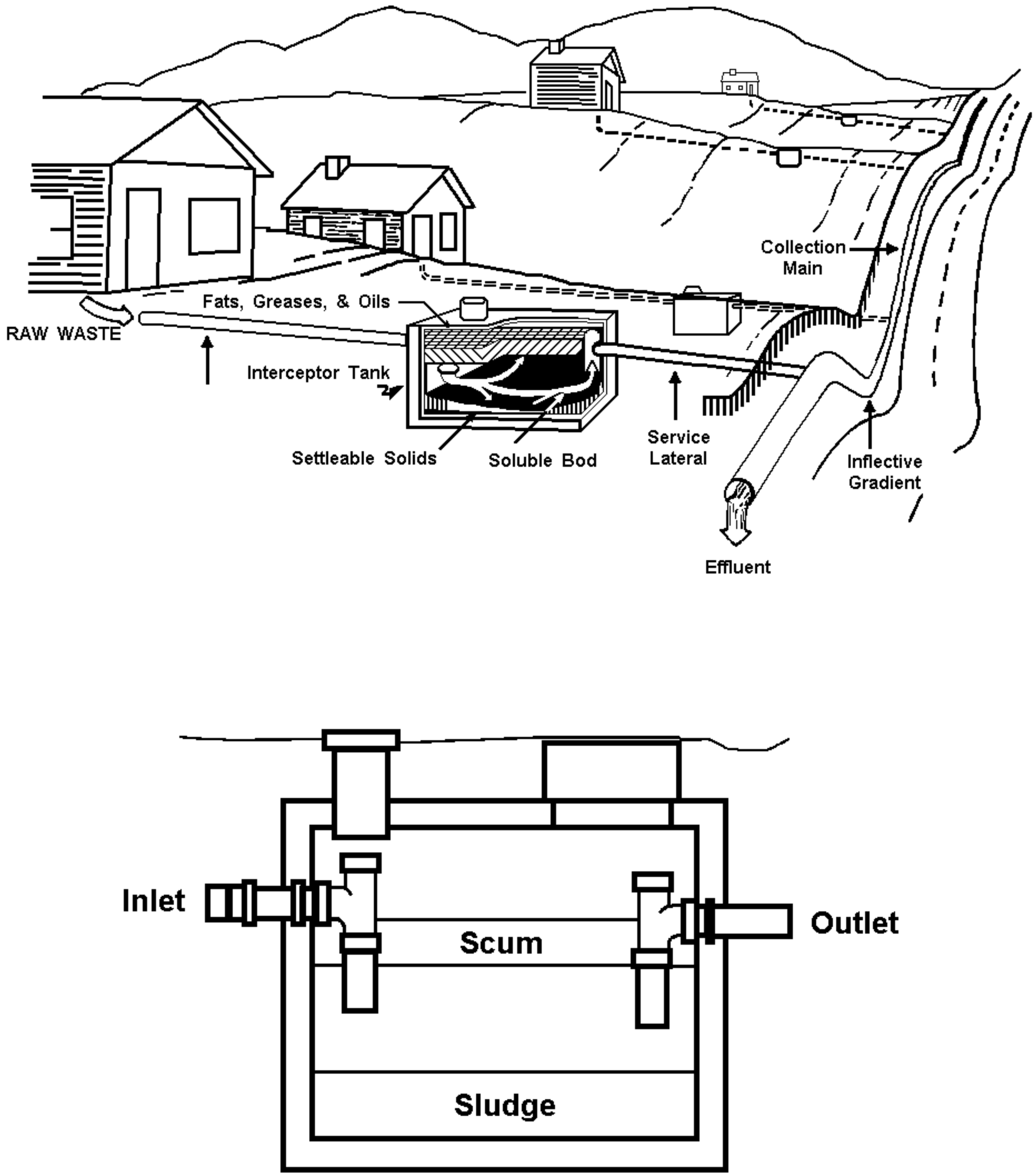


Figure 6-9. Components of small diameter gravity sewer (SDGS) system (USEPA 1991b).

Composting

On-site composting has been attempted at an ecovillage in Sweden (Fittschen and Niemczynowicz 1997). Toilet wastes were deposited in an on-site composting tank. The results of this experiment were less than desirable for a variety of reasons. The system is user-intensive, demanding a level of expertise beyond that of average residents. Technically it was only partially acceptable because the resulting compost was only of mediocre quality for agricultural use. The system was found to be socially unacceptable and was energy intensive as electricity was used to dry the compost (Fittschen and Niemczynowicz 1997). Again, this technology is in the testing phase, though it may hold promise for specific applications.

Combined Systems for the Future?

While old CSS are considered a major source of urban pollution, there is some recent activity in the area of new CSS. Where urban areas have significant amounts of NPS pollution that requires treatment, it may be possible to design a CSS to capture most of the annual storm volume for treatment at a WWTP, without discharging raw sewage during major events. Lemmen et al. (1996) describe a concept for a sewer system in the Netherlands that has connections between the storm drainage network and the sanitary collection system.

Walesh and Carr (1998) and Loucks and Morgan (1995) describe use of controlled storage of stormwater on and below streets to control surcharging and solve basement flooding in a CSS. The premise of this approach is that stormwater flow rate, not volume, is the principal cause of surcharging of CSS and resulting basement flooding and CSO. On and below street storage of stormwater, strategically placed throughout the CSS, reduces peak stormwater flows to rates that can be accommodated in the CSS without surcharging. The two large scale, constructed, and cost effective projects described by Walesh and Carr and by Loucks and Morgan were retrofits. However, the success of these projects suggests integrating the design of streets and CSS in newly developing high intensity areas.

Future Directions: Collection Systems of the 21st Century

New ideas for managing the entire urban water cycle in an integrated fashion are being formulated. This section synthesizes various aspects of recent research into a vision of what the near future may hold for collection systems in the 21st century. In order to synthesize these ideas, probable contextual factors within which collection systems will operate must be examined.

The definitive settlement type of the second half of the 20th century in the U.S. has been urban sprawl. In the U.S., this land use has been brought on largely by zoning and the proliferation of the automobile. Recent ideas regarding resource allocation seem to indicate that, while the automobile is not likely to disappear in this country in the next 50 years, its function may change. The “new urbanism” is likely to have mixed land use areas typified by neighborhoods where specific land use types may not dominate a specific urban catchment. Neighborhoods replace zoned land use types in the new urbanism and, as such, present a variety of opportunities for innovative urban water management.

The main premise of this discussion is that new urban development in the 21st century will begin to follow the patterns of the “new urbanism” in terms of land use and transportation. The other guiding premise is that design will be control-driven, that is to say that new systems will be designed to be controlled far beyond that which is presently used in wet-weather management. Therefore, the following scenarios describe possible future collection systems for new urban areas that integrate source control, system control, data management, life-cycle costs, environmental costs, and social acceptability.

Future Collection System Scenarios

High Density Areas

Areas with the highest levels of urban NPS will require stormwater treatment, much as they do today. A form of CSS, or an integrated storm-sanitary system (ISS) (Lemmen et al. (1996), will capture a large portion of the annual runoff volume from dense urban areas. Storm runoff will be reduced by source control and infiltration BMPs and the residual of small events will be transported to the WWTP. Large events will be throttled out of the ISS, before mixing with sanitary waste, and discharged to receiving waters. This new system will have the best of both CSS and separate systems. The advantage of the combined system has been treatment of small runoff producing events, including snowmelt. However, the disadvantage has always been the discharge of raw sewage to receiving waters during large events. With the advantage of control technology, as the sewers and/or the WWTP reach capacity, the stormwater is diverted directly to receiving waters, without mixing with sanitary and industrial wastes.

This system will have a high degree of built in control. The data stream begins with local radar observations. This information is combined with real-time ground level measurements of rainfall. These data will be used to predict the rainfall patterns over the catchment for the next half hour. The SCADA system receives information regrading the present state of the sanitary and storm portions of the waste stream. Quality as well as quantity are monitored. Performance of high rate treatment devices operating on the discharged stormwater is monitored. A critical innovation is the integration of the WWTP performance, operation and control into the system. Operation of the WWTP now extends to the collection system. Rainfall information in conjunction with the state of the system and receiving water data are used to predict potential outcomes of the wet-weather event using a system simulation model. Coupled with a non-linear optimization routine, an optimal control scheme is determined on-line and changes in system control are relayed back to the system via the SCADA system.

The system response is fed back to the SCADA and continuous control is maintained throughout the wet-weather event. This “feedback” loop provides the municipality with rapid response for flashy summer events and provides urban flood control simultaneously with water quality control.   In addition, the time series of wet-weather data are now stored in a relational database, spatially segregated to interface with static geography stored in a GIS.

Suburban Development

Outlying from the new urban centers, suburban type development still exists. While less dense than the city, new suburban development contains some of the mixed land uses found in the urban center. The collection system serving this area is far different from the city, however, because the NPS pollution is not so severe as to warrant full treatment at the WWTP. BMPs and source control innovations have reduced the stormwater impacts on the receiving water. Regional detention is used for flood control and water quality enhancement while possibly providing recreation.

Sanitary wastes are transported via pressure sewers to collector gravity lines at the city's border. The use of pressure sewers has reduced suburban I/I to near zero. In addition, the new sanitary LPS sewers are very easy to monitor, as the age-old problem of open channel flow estimation is avoided by using pressure lines. This provides added certainty in the flow estimation and lends itself very well to control. Technology borrowed from the water distribution field has achieved a great level of system reliability and control. In fact, the sewer now mirrors the water distribution network, essentially providing the inverse service.

References

Ackers, J., D. Butler, S. John and R. May (1996). Self-Cleansing Sewer Design: The CIRIA Procedure. In Sieker, F. and H.R. Verworn (Ed.). Proceedings of the Seventh Annual Conference on Urban Storm Drainage. Hannover, Germany. September 9-13, 1996. Vol. II. IAER/IAWQ Joint Committee on Urban Storm Drainage.

American Society of Civil Engineers and Water Pollution Control Federation (1982). Gravity Sanitary Sewer Design and Construction. ASCE No. 60. WPCF No. FD-5. ASCE. New York, NY. WPCF. Washington DC.

American Society of Civil Engineers and Water Environment Federation (1993). Design and Construction of Urban Stormwater Management Systems. ASCE No. 77. WEF No. FD-20. ASCE. New York, NY. WEF. Washington DC.

Ashley, R.M. (Ed.) (1996). Solids in sewers. Water Science & Technology. Vol. 33. No. 9. Association of Metropolitan Sewerage Agencies (AMSA). (1996).

Ashley, R.M. and Verbanck, M.A. (1997) Physical Processes in Sewers. In Emscher Congress: 'Water Management in Conurbations' June 19 and 20, 1997, Bottrop, Germany.

Association of Metropolitan Sewerage Agencies (AMSA) (1996). The AMSA Financial Survey, 1996, A National Survey of Municipal Wastewater Management Financing Trends. AMSA. Washington, DC.

Bally, D., T. Assano, R. Bhamidimarri, K.K. Chin, W.O.K. Grabow, E. R. Hall, S. Ohgaki, D. Orhon, A. Milburn, C. D. Purdon and P. T. Nagle. (Ed.) (1996). Water Quality International '96 Part 2. Water Science & Technology. Vol. 34. No. 3-4.

Chow, V.T. (1959). Open Channel Hydraulics. McGraw-Hill Book Co. New York, NY.

Clemente, A.J. and R.W Cardozo (1996). Dade county's mandated improvement program to reduce sanitary sewer overflows. In USEPA. National Conference on Sanitary Sewer Overflows (SSOs). EPA/625/R-96/007. Office of Water. Washington, DC.

Collins, E. (Ed.) (I 994). On-site wastewater treatment. Proceedings of the Seventh Annual International Symposium on Individual and Small Community Sewage Systems. Atlanta, GA. December 11-13, 1994. American Society of Agricultural Engineers. St. Joseph, MI.

Dent, S.D, and D. Davis (1995). Database management model for SCADA systems. In James, W (Ed.). Modem Methods for Modeling the Management of Stormwater Impacts. Computational Hydraulics International. Guelph, Ontario, Canada.

DeOreo, W., J. Heaney and P. Mayer (1996). Flow trace analysis to assess water use. Journal of the American Water Works Association. Jan., p. 79-80.

Farrell, R.P., and G. Grey Darrah (1994). Pressure sewers -- a proven alternative solution for a variety of small community sewage disposal challenges. In Collins, E. (Ed.). OnSite Wastewater Treatment. Proceedings of the Seventh Annual International Symposium on Individual and Small Community Sewage Systems. Atlanta, GA. American Society of Agricultural Engineers. St. Joseph, MI.

Fittschen, I., and J. Niemczynowicz (1997). Experiences with dry sanitation and greywater treatment in the ecovillage Toarp, Sweden. In Henze, M., L. Somlyody, W. Schilling, and J. Tyson (Ed). Sustainable Sanitation. Water Science & Technology. Vol. 35, No. 9. Elsevier Science Ltd. Oxford, UK.

Foil, J. L., J. A. Cerwick, and J. E. White (1993). Collection systems past and present. Water Environment and Technology. December.

Field, R. (1990). Combined sewer overflows: control and treatment. In Moffa, P.E. Control and Treatment of Combined Sewer Overflows. Van Nostrand Reinhold. New York, NY.

Gayman, M. (1996). A glimpse into London's early sewers. Cleaner Magazine. Cole Publishing Inc. Three Lakes, WI.

Gent, R., B. Crabtree and R. Ashley (1996). A review of model developments based on sewer sediments research in the UK. In Ashley, R. (Ed.). Solids in Sewers. Water Science & Technology. Vol. 33. No. 9.

Gonwa, W., N. Schultz, L. Paulus, and V. Novotny (1994). Improved collection system real time control for Milwaukee MSD. In WEF. WEFTEC '94 Proceedings of the Water Environment Federation 67th Annual Conference and Exposition. Chicago, IL. October 15-19, 1994. Part 1: Collection Systems. volume 111. Water Environment Federation. Alexandria, VA.

Gross, C.E., N. Huang, J.T. Mauro and E.D. Driscoll (1994). Nine minimum control requirements for combined sewer overflows. In WEF. A Global Perspective for Reducing CSOS: Balancing Technologies, Costs, and Water Quality. WEF Specialty Conference Series Proceeding. July 10- 13, 1994. Louisville, KY. Water Environment Federation. Alexandria, VA.

Hanaeus, J., D. Hellstrom and E. Johansson (1997). A study of a urine separation system in an ecological village in northern Sweden. In Henze, L., Somlyody, W. Schilling and J. Tyson (Ed.). Sustainable Sanitation. Water Science & Technology. Vol. 35. No. 3. Elsevier Science Ltd. Oxford, UK.

Harremoes, P. (1997). Integrated water and waste management. In Henze, L., Somlyody, W. Schilling and J. Tyson (Ed.). Sustainable Sanitation. Water Science & Technology. Vol. 35. No. 3. Elsevier Science Ltd. Oxford, UK.

Hassett, A. (1995). Vacuum sewers -- ready for the 21st Century. In WEF. Sewers of the Future. WEF Specialty Conference Series Proceeding. September 10-13, 1995. Houston, TX. Water Environment Federation. Alexandria, VA.

Heaney, J., D. Sample and L. Wright (1997). Class notes, EPA project meeting and modeling workshop. March 24-26, (1997). Environmental Protection Agency. Edison, NJ.

Heaney, J. and L. Wright (1997). On integrating continuous simulation and statistical methods for evaluating urban stormwater systems. In James, W. (Ed.). Advances in Modeling the Management of Stormwater Impacts. Vol. 5. Computational Hydraulics International. Guelph, Ontario, Canada.

Henze, M., L. Somlyody, W. Schilling and J. Tyson (Ed.) (1997). Sustainable sanitation. Water Science & Technology. Vol. 35. No. 3. Elsevier Science Ltd. Oxford, UK.

James, W. (Ed.) (1997). Advances in modeling the management of stormwater impacts. Vol. 5. Computational Hydraulics International. Guelph, Ontario, Canada.

James, W. (Ed.) (1995). Modem Methods for Modeling the Management of Stormwater Impacts. Computational Hydraulics International. Guelph, Ontario, Canada.

Jeng, K., M.J. Bagstad, and J. Chang (1996). New collection system modeling techniques used in Houston. In USEPA. National Conference on Sanitary Sewer Overflows (SSOs). EPA/625/R-96/007. Office of Water. Washington, DC.

Jurgens, D.E. and H.M. Kelso (1996). Sewer rehabilitation: the techniques of success. In USEPA. National Conference on Sanitary Sewer Overflows (SSOs). EPA/625/R96/007. Office of Water. Washington, DC.

Lemmen, G., D. de Bijil, and M. Maessen (1996). A new development in the City of Dordrecht: sewerage and drainage masterplan of Buitenstad. In Sieker, F. and H.R. Verworn (Ed.). Proceedings of the Seventh Annual Conference on Urban Storm Drainage. Hannover, Germany. September 9-13, (1996). Vol. II. IAHR/IAWQ Joint Committee on Urban Storm Drainage.

Loucks, E. D. and M. G. Morgan (1995). Evaluation of the Wilmette runoff control program. Proceedings of Integrated Water Resources Planning for the 21st Century. American Society of Civil Engineers. New York, NY. May.

Mara, D. (1996). Low Cost Urban Sanitation. Wiley. New York, NY

Mark, O., U. Cerar and G. Perrusquia (1996). Prediction of locations with sediment deposits in sewers. In Ashley, R. (Ed.). Solids in Sewers. Water Science & Technology. Vol. 33. No. 9.

Merril, M. Steve and R. Butler (1994). New dimensions in infiltration/inflow analysis. In WEF. A Global Perspective for Reducing CSOs: Balancing Technologies, Costs, and Water Quality. Water Environment Federation Specialty Conference Series Proceedings. July 10- 1 3, 1994. Louisville, KY. WEF. Alexandria, VA.

Metcalf, L., and H. P. Eddy (1914). American Sewerage Practice: Volume I Design of Sewers. Mcgraw-Hill. New York, NY.

Miles, S. W., J. L. Dom, and R.E. Tarker Jr. (1996). An I/I analysis and prediction method to help guide separate sanitary sewer improvement programs. In WEF. Urban Wet Weather Pollution: Controlling Sewer Overflows and Stormwater Runoff. WEF Conference Proceedings. Quebec City, Canada. June 16-19, 1996. WEF. Alexandria, VA.

Moffa, P. E. (Ed.) (1990). Control and Treatment of Combined Sewer Overflows. Van Nostrand Reinhold. New York, NY.

Petroff, R.G. (1996). An analysis of the root causes of SSOs. In USEPA. National Conference on Sanitary Sewer Overflows (SSOs). EPA/625/R-96/007. Office of Water. Washington, DC.

Pisano, W.C. (1996). Summary: United States "sewer solids" settling characterization methods, results, uses and perspectives. In Ashley, R. (Ed.). Solids in Sewers. Water Science & Technology. Vol. 33. No. 9.

Read, G. F., and I. G. Vickridge (Ed.) (1997). Sewers - Rehabilitation and New Construction, Repair and Renovation. Arnold, London, UK.

Rudolph, R.S., E.T. Kelly and J.D. Sharon (1995). A guide to efficiently isolate I/I sources related to basement flooding occurrences. In WEF. Sewers of the Future. WEF Specialty Conference Series Proceeding. September 10-13, 1995. Houston, TX. Water Environment Federation. Alexandria, VA.

Schafer, P. L. (1994). Two feet per second ain't even close. In WEF. WEFTEC '94: Collection Systems and Residuals & Biosolids Management. Proceedings of the Water Environment Federation 67th Annual Conference and Exposition. Chicago, IL. October 15-19, (1994). WEF. Alexandria, VA.

Sieker, F. and H. R. Verworn (Ed.) (1996). Proceedings of the Seventh Annual Conference on Urban Storm Drainage. Hannover, Germany. September 9-13, (1996). Vols. 1, II and III. IAHR/IAWQ Joint Committee on Urban Storm Drainage.

Tchobanoglous, G. (1981). Wastewater Engineering: Collection and Pumping of Wastewater. McGraw-Hill. New York, NY.

United States Army Corps of Engineers (USACE) (1994). Risk-Based Analysis for Evaluation of Hydrology/Hydraulics and Economics in Flood Damage Reduction Studies. Circular No. 1105-2-205. Department of the Army. Washington, DC.

USEPA (1990). Rainfall Induced Infiltration into Sewer Systems. Report to Congress. EPA/430/09-90/005. Office of Water. Washington, DC.

USEPA (1991a). Sewer System Infrastructure Analysis and Rehabilitation. EPA/625/6-91/030. Office of Water. Washington DC.

USEPA (1991b). Manual: Alternative Wastewater Collection Systems. EPA/625/1-91/024. Office of Water. Washington DC.

USEPA (1993). Manual: Combined Sewer Overflow Control. EPA/625/R-93/007. Office of Water. Washington, DC.

USEPA (1995a). Combined Sewer Overflows: Guidance For Long-Term Control Plan. EPA 832-B-95-002. Office of Water. Washington, DC.

USEPA (1995b). Combined Sewer Overflows: Guidance For Nine Minimum Controls. EPA 832-B-95-003. Office of Water. Washington, DC.

USEPA (1995c). Combined Sewer Overflows: Screening and Ranking Guidance. EPA 832-B95-004. Office of Water. Washington, DC.

USEPA (1996a). Combined Sewer Overflows: Guidance For Monitoring And Modeling. EPA 832-B-97-001 Draft. Office of Water. Washington, DC.

USEPA (1996b). National Conference on Sanitary Sewer Overflows (SSOs). EPA/625/R96/007. Office of Water. Washington, DC.

Vitasovic, Z., J. Dumont, R. Fitzgerald and J. Mackenzie (1994). Comprehensive approach to real time control of CSOs. In WEF. A Global Perspective for Reducing CSOs: Balancing Technologies, Costs, and Water Quality. WEF Specialty Conference Series Proceeding. July 10-13, 1994. Louisville, KY. Water Environment Federation. Alexandria, VA.

Walesh, S.G. and R. W. Carr (1998). Controlling stormwater close to the source: an implementation case study. American Public Works Congress. Las Vegas, NV. September.

Walker, D.E., J.B. Golden, D.R. Bingham, E.D. Driscoll, and D.J. Murray (1994). Design considerations for off-line, near surface storage/treatment facilities. In WEF. A Global Perspective for Reducing CSOs: Balancing Technologies, Costs, and Water Quality. WEF Specialty Conference Series Proceeding. July 10-13, 1994. Louisville, KY. Water Environment Federation. Alexandria, VA.

Water Environment Federation (WEF) (1994a). A Global Perspective for Reducing CSOs: Balancing Technologies, Costs, and Water Quality. WEF Specialty Conference Series Proceeding. July 10-13, 1994. Louisville, KY. Water Environment Federation. Alexandria, VA.

WEF (1994b). WEFTEC '94 Proceedings of the Water Environment Federation 67th Annual Conference and Exposition. Chicago, IL. October 15-19, 1994. Part 1: Collection Systems. Volume III. Water Environment Federation. Alexandria, VA.

WEF (1995a). Sewers of the Future. WEF Specialty Conference Series Proceedings. September 10-13, 1995. Houston, TX. Water Environment Federation. Alexandria, VA.

WEF (1995b). WEFTEC '95 Proceedings of the Water Environment Federation 68th Annual Conference and Exposition. Miami Beach, FL. October 21-25, 1995. WEF. Alexandria, VA.

WEF (1996). Urban Wet Weather Pollution, Controlling Sewer Overflows and Stormwater Runoff Proceedings. June 16-19, 1996. Quebec City, Quebec, Canada. WEF. Alexandria, VA.

WEF/ASCE (1994). Existing Sewer Evaluation & Rehabilitation. WEF FD-6. ASCE No. 62. Water Environment Federation. Alexandria, VA.

Zukovs, G., D. Cuthbert, W. Pisano, M. Umberg, and T. Quinn (1996). Watershed based CSO planning in greater Cincinnati. In Sieker, F. and H.R. Verwom (Ed.). Proceedings of the Seventh Annual Conference on Urban Storm Drainage. Hannover, Germany. September 9-13, 1996. IAHR/IAWQ Joint Committee on Urban Storm Drainage.

Chapter 7

Assessment of Stormwater Best Management Practice Effectiveness

Ben Urbonas

Introduction

The use of stormwater practices to control and manage the quality and quantity of urban runoff has become widespread in U.S. and in many other countries. As a group they have been labeled as best management practices or BMPs. Current literature describes a variety of techniques to reduce pollutants found in separate urban stormwater runoff (that is, not CSS). Many of these same practices can also be applied for areas served by CSS to reduce the frequency of combined sewer overflows (CSOs) during wet weather and to enhance quality of the CSOs when they do occur.

Structural BMPs are designed to function without human intervention at the time wet weather flow is occurring, thus they are expected to function unattended during a storm and to provide passive treatment. Nonstructural BMPs as a group are a set of practices and institutional arrangements, both with the intent of instituting good housekeeping measures that reduce or prevent pollutant deposition on the urban landscape.

Much is known about the technology behind these practices, much is still emerging and much remains yet to be learned. Currently many of these controls are used without full understanding of their limitations and their effectiveness under field (i.e., real world) conditions, as opposed to regulatory expectations or academic predictions or beliefs. In addition, the uncertainties in the state of practice associated with structural BMP selection, design, construction and use are further complicated by the stochastic nature of stormwater runoff and its variability with location and climate. Where one city may experience six months of gentle, long-duration rains; another will experience many convective and frontal rainstorms followed by severe winter snows that melt in the spring; while still another will experience few, mostly convective storms. At the same time, examination of precipitation records throughout the U.S. reveals that the majority of individual storms are relatively small, often producing less precipitation and runoff than used in the design of traditional storm drainage networks.

A number of structural and non-structural BMPs are discussed in this chapter focusing on their effectiveness in removing pollutants and in mitigating flow rates. BMP effectiveness in addressing some of the stipulated impacts of urban runoff on receiving water systems is also discussed.

After much literature review Roesner, Urbonas and Sonnen (1989) concluded the following:

> ***Among all these devices the most promising and best understood are detention and extended detention basins and ponds. Less reliable in terms of predicting performance, but showing promise, are sand filter beds, wetlands, infiltration basins, and percolation basins. All of the latter appear to be in their infancy and lack the necessary long-term field testing that would provide data for the development of sound design practices.***

Information published since 1989 has expanded very little understanding of structural BMPs and their performance. However, urban water professionals may be on a verge of a breakthrough in identifying and possibly quantifying some of the linkages between the urban runoff processes and its effects on various aspects of receiving systems. This should lead to a better understanding of how and why various types of BMPs may be able to moderate some of the effects on receiving systems. It is unlikely, however, that BMPs and other techniques will be able to eliminate all of the effects on receiving systems that are caused by the growth in population world wide, especially the population growth of urban areas.

Objectives in the Use of Best Management Practices for Stormwater Quality Management

The comprehensive -- quantity and quality -- approach to stormwater management is relatively new. Prior to the late 1960's the primary goal was to rapidly drain municipal streets and to convey this drainage to the nearest natural waterway. This practice evolved into the use of detention when the municipal engineers began to recognize that the cost of urban drainage systems became prohibitive as more and more of the watershed urbanized. Also, some began to recognize the deleterious effects that uncontrolled urban drainage had on the stability of the receiving stream. One of the first states to require the control of smaller runoff events, namely the peak runoff rate from the two-year design storm, was Maryland. In the late 1970's, Maryland was also the first to require stormwater quality BMPs, including stormwater infiltration. As a result, it and some of the other states like Florida became early field test beds for these facilities. Although much has yet to be learned before engineers can design for a specific performance, BMP knowledge is evolving. Currently, the design professional and the planner have to think in terms of how to best manage stormwater runoff in order to limit damage to downstream properties, reduce stream erosion, limit the effects on the flora and fauna of the receiving streams and integrate stormwater systems into the community.

As the field of stormwater management expanded in its scope, water quality became an increasingly important consideration at many locations in the U.S. Structural BMPs cannot do the job alone without the cooperation and participation of the public. Prevention and good housekeeping became two operative words and practices. They are now considered as important as the use of structural BMPs and may be the only affordable approaches for much of the currently urbanized landscape.

Figure 7-1 conceptually summarizes four basic objectives for stormwater quality management. The first objective includes the concepts of prevention and load reduction. This is followed by the use of other non-structural and structural measures.

The following four objectives provide an integrated and balanced approach to help mitigate the changes in stormwater runoff flows that occur as land urbanizes and to help mitigate the impacts of stormwater quality on receiving systems:

1. Prevention: Practices that prevent the deposition of pollutants on the urban landscape including changes in the products that, when improperly used or accidentally spilled, deposit pollutants on the urban landscape and changes in how the public uses and disposes of these types of products.

2. Source control: Preventing pollutants from coming into contact with precipitation and stormwater runoff.

3. Source disposal and treatment: Reduction in the volume and/or rate of surface runoff and in the associated constituent loads or concentrations at, or near their source.

4. Follow-up treatment: Interception of runoff downstream of all source and on-site controls using structural BMPs to provide follow-up flow management and/or water quality treatment.

Whenever two or more of these objectives are implemented in series within a watershed, they form a treatment train. A long line of discussions among some regulators and stormwater professionals indicates a belief that the implementation of more than one of these objectives in a treatment train fashion (Livingston et al., 1988, Roesner et al. 1991, Schueler et al.,1991, Urbonas and Stahre 1993, WEF & ASCE 1998) will result in better quality stormwater reaching the receiving waters. Whether this is true or not has not been conclusively field tested. Intuitively this assertion makes sense, but whether the use of a set of structural BMPs or the use of more than one of these objectives in various combinations has any significant or measurable mitigation of urban runoff effects on the receiving waters has yet to be answered. Obtaining the answer will require well designed and controlled field studies, with each taking place over a number of years. Nevertheless, each set of practices appears to add to the arsenal of tools that help manage stormwater runoff and its quality. If nothing else, their use probably adds to the quality of urban life and the enjoyment of the receiving waters into which urban runoff drains.

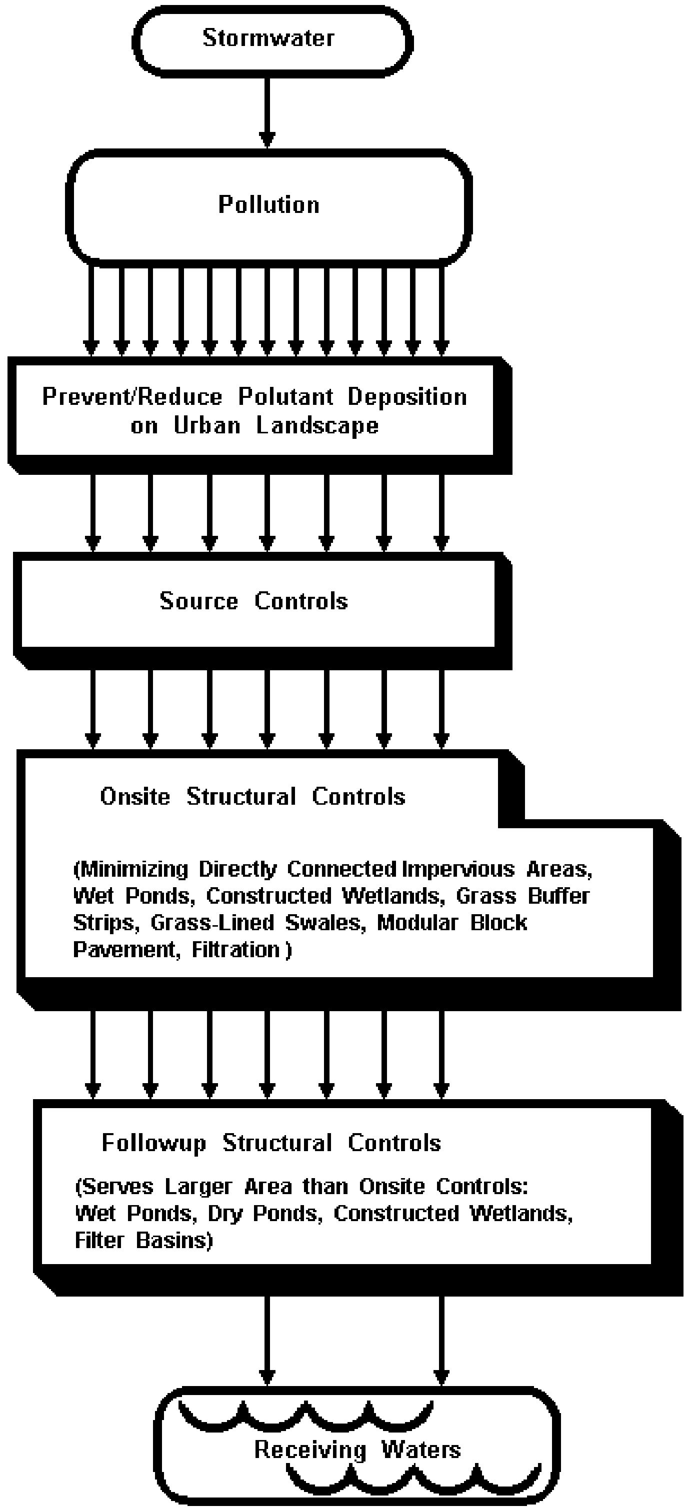


Figure 7-1. BMPs in series to minimize urban stormwater runoff quality impacts (UD&FCD 1992).

Non-Structural Best Management Practices

Non-structural BMPs include a variety of institutional and educational practices that, hopefully, result in behavioral changes which reduce the amount of pollutants entering the stormwater system and, eventually, the receiving waters into which it drains. Some of these non-structural practices deal with the land development and redevelopment process. Others focus on educating the public to modify behavior that contributes to pollutant deposition on urban landscapes. Others search out and disconnect illicit wastewater connections, control accidental spills, and enforce violations of ordinances designed to prevent the deposition of pollutants on the urban landscape and its uncontrolled transport downstream. Among a variety of practices, non-structural BMPs include:

1. Discontinuing or reducing the use of products that have been identified as a problem (e.g., use of phosphorous free or low phosphorous detergents, limiting the application of pesticides, calibrating the application of sand and salt applicators to road surfaces in winter).

2. The adoption and implementation of building and site development codes to encourage or require the installation of structural BMPs for a new development and significant redevelopment projects.

3. Adoption and implementation of site disturbance/erosion control programs.

4. Minimizing the DCIA in new development, including the use of landscaped areas for the discharge of stormwater from impervious surfaces, grass buffers, and roadside swales instead of curb and gutter.

5. Public education on the proper uses and disposal of potential pollutants such as household chemicals, paints, solvents, motor oils, pesticides, herbicides, fertilizers, and antifreeze.

6. Effective street sweeping and leaf pickup and efficient street deicing programs.

7. Detection and elimination of illicit discharges from wastewater lines to separate storm sewers.

8. Enforcement of the operation and maintenance requirements of privately owned stormwater management facilities, including on-site structural BMPs and non-structural programs.

9. Providing the needed operation and maintenance for publicly owned BMPs.

Structural Best Management Practices

Stormwater runoff quality enhancement begins with the avoidance and prevention of pollutant deposition onto the urban landscape (Urbonas and Stahre 1993). It is likely that structural BMPs cannot do the job alone and be fully effective. Structural BMPs need to be viewed as only a supplement to the "good housekeeping measures" being practiced within a community. Once the development and implementation of a non-structural program is in progress, the use of the BMPs discussed in this section can be considered.

Minimized Directly Connected Impervious Area

This practice is listed under the structural BMPs because it can be provided only when land is being developed (i.e., changed from agricultural or an undeveloped state to an urban development) and when significant amounts of older urbanized areas undergo redevelopment. Retrofitting this BMP into developed areas is probably not generally feasible because of the great expense and the physical disruption of neighborhoods and their residents.

Minimizing DCIA relies on the construction of urban streets, parking lots and buildings using a non-traditional template. Figure 7-2 illustrates two hypothetical areas, one using traditional drainage practices and the other the minimal DCIA concept. Instead of elevated landscape islands in a commercial areas, this concept uses landscaped areas that are lower than the adjacent street and parking lot grades to intercept, detain and convey surface runoff. Also, porous pavement parking pads can be used to intercept surface runoff from impervious paved areas. This concept for new land development includes an extensive use of swales, grass buffer strips, porous pavement, and random placements of infiltration basins (infiltration areas) whenever site conditions permit. Not all of the features illustrated in Figure 7-2 are feasible at all sites, nor is this concept feasible for all development sites or land use types. Site conditions such as local geology, soils, groundwater levels, terrain slopes, soil stability, meteorology, land uses and development policies need to be fully evaluated to determine if this practice is feasible.

The intent is to slow down the rate of stormwater runoff and to encourage infiltration. In so doing, surface runoff volumes during small storms can be reduced somewhat for the majority of sites and totally eliminated under most favorable site conditions.

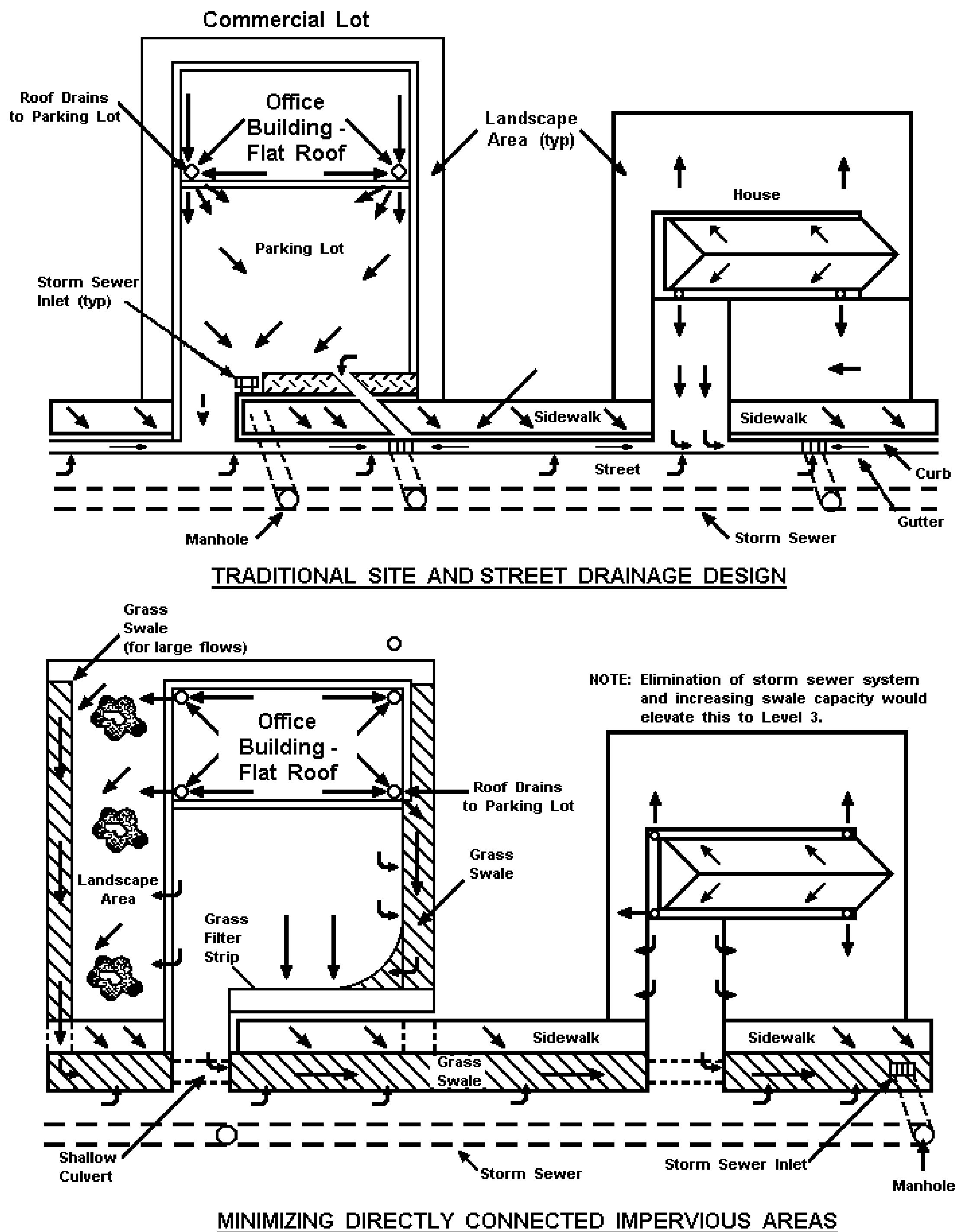


Figure 7-2. Comparing traditional and minimized directly connected impervious area drainage (UD&FCD 1992).

Water Quality Inlets

Water quality inlets are single or multi-chambered underground sediment or sediment and oil separation vaults. Some are simple catch basins with a depressed bottom where the heavier sediments settle before stormwater enters the downstream conveyance system. Others are more complex, equipped with more than one chamber, have lamella plates and/or are designed to separate solids, floatables, oils and greases from water. These type of devices have been in use for years and primarily serve very small tributary catchments.

Infiltration Practices

This group of structural BMPs include swales, grass buffer strips, porous pavement, percolation trenches, and infiltration basins. Water that infiltrates can sometimes drain to the groundwater table. As a result, this practice has to be used with caution and may not be appropriate for sites that have gasoline stations, chemical storage areas and other activities that that can contaminate land surfaces and the groundwater below. Each of these practices is described in more detail as follows:

1. Grass Swale: The slower the flow in a grass swale, the more pollutants will be removed from stormwater through sedimentation and the straining of surface runoff through the vegetative cover. Also, the slower the flow, the more time stormwater has to infiltrate into the ground. The ultimate in slow flow is a swale that acts as a linear detention basin.

2. Grass buffer strip: To remove the heavier sediment particles, a grass buffer strip has to have a flat surface with a healthy turf-forming grass cover. Pollutants are removed from stormwater primarily through sedimentation and the straining of stormwater runoff through the vegetative cover. In arid and semi-arid climates, grass buffer strips need to be irrigated (UD&FCD 1992).

3. Porous Pavement: Porous pavement has been used in the U.S. and Europe since the mid-1970s. It is constructed either of monolithically poured porous asphalt or concrete, or modular concrete paver blocks.

4. Percolation Trench: A percolation trench is a rock filled trench that temporarily stores stormwater and percolates it into the ground. A percolation trench typically serves small impervious tributary areas of two hectares or less.

5. Dry Well: A dry well is a rock filled vertical well that temporarily stores stormwater in order to allow it time to percolate into the ground. It is similar in operation to a percolation trench. Dry wells are sometimes used to penetrate an impermeable layer near the surface to provide a stormwater conduit to a permeable soil layer that lies below it. Dry wells typically serve small impervious tributary areas of two hectares or less.

6. Infiltration Basin: An infiltration basin intercepts and temporarily stores stormwater on its surface, where it eventually infiltrates into the ground. An infiltration basin often serves a small developed catchment, one with less than four hectares of tributary impervious surface.

Filter Basins and Filter Inlets

The use of media filter basins, mostly sand filters, for stormwater quality enhancement was first reported by Wanielista et al. (1981) and Veenhuis et al. (1988). Since then the use of filters has expanded, with most uses reported in the State of Delaware, the Washington DC area, Alexandria, VA and the Austin, TX area (City of Austin 1988, Livingston et al. 1988, Anderson et al. Undated, Chang et al. 1990, Truong et al. 1993, Bell et al. 1996).

Recently, media filters such as peat-sand mix, sand-compost mix and goetextiles have also been tested and proposed for use (Farham and Noonan 1988, Galli 1990, Stewart 1989). An ingenious sand filter inlet has been suggested by Shaver and Baldwin (1991). In most of the suggested filter designs, a detention volume is provided upstream of the filter media. This volume captures the runoff and permits it to flow through the filter at a flow rate compatible with its size and hydraulic conductivity.

Swirl-Type Concentrators

These complex underground vaults are designed to create circular motion within the chamber to encourage sedimentation and the removal of oil and grease. They are also often equipped with trash skimmers and traps. Swirl concentrators are designed to effectively process up to a design flow rate and to by-pass higher flow rates.

Extended Detention Basins

Detention basins hold stormwater temporarily (i.e., detain). They are sometimes called dry detention basins or ponds because they drain out, for the most part, completely after the runoff from a storm ends and then they remain “dry” until the next runoff event begins. The joint use of the terms “dry-pond” is an oxymoron and, for the sake of consistent terminology, the expression detention basin is suggested.

Retention Ponds

Retention ponds have a permanent pool. Some are equipped with a formal surcharge detention volume above this pool. Processes that are known, or are suspected to be at work in a retention pond are sedimentation, flocculation, agglomeration, ion exchange, adsorption, biological uptake through microbial and plant ingestion and eutrophication, remobilization, solution, and physical resuspension of particulates. In the main body of the pond, particulate pollutants are removed by settling and nutrients are removed by phytoplankton, algal and bacterial growth in the water column. Marsh plants around the perimeter of the pond provide the biological media to help remove nutrients and other dissolved constituents and trap small sediment and algae in the water column.

Wetlands

Currently, the use of wetlands as stormwater quality enhancing facilities is an emerging technology. Wetlands can be used as source controls or as follow-up treatment devices. A wetland basin, in essence, is another form of an extended detention basin or a retention pond. As a result, all of the constituent removal processes listed for an extended detention basin and a retention pond should also apply to a wetland basin.

A wetland channel is similar to a grass-lined channel, except it is designed to develop wetland growth on its bottom and is typified by a flat longitudinal slope, wide bottom and slow flow velocities during the two-year and smaller storm runoff events. A wetland channel, to a smaller degree and depending on specific site conditions and design, probably has many of the constituent removal characteristics of a wetland basin.

Stormwater Quality Management Hydrology

Urbonas, Guo and Tucker (1990) observed that capture volume effectiveness in the Denver, CO area reached a point of diminishing returns. This point, referred to by some as the "knee of the curve," was later defined as the point of maximized capture volume (Urbonas and Stahre 1993). Figure 7-3 indicates that this is the point where rapidly diminishing returns begin to occur. Beyond this point the number of events and the total volume of stormwater runoff fully captured during an average year decrease significantly as the detention volume is increased.

Although the number of storms, and their characteristics such as intensity, volume, duration, seasons, and storm separation vary with location, a pattern of diminishing returns was observed by Roesner et al (1991), Guo and Urbonas (1996) Urbonas et al (1996 a), Heaney and Wright (1997) and others. This seems to be the case for all precipitation gauging sites analyzed, regardless of the hydrologic regions in U.S. in which they are located. The other finding was that the maximized capture volume, once determined for a given site, captured 80 to 90% of all runoff events and runoff volumes at the site. This volume was also sufficient to capture the “first flush” of storm runoff during the larger events that exceed the design capture volume.

Table 5.1 in WEF & ASCE (1998) lists the maximized capture volumes at six study sites studied by Roesner et al. (1991) located in different hydrologic regions of U.S. They observed that 1.0 watershed inches (25.4 mm) of storage volume captured more than 90% of all the runoff volume at all six sites and that 0.5 watershed inches (13 mm) of available storage volume captured over 90% of the runoff at the four residential neighborhoods among the six sites.

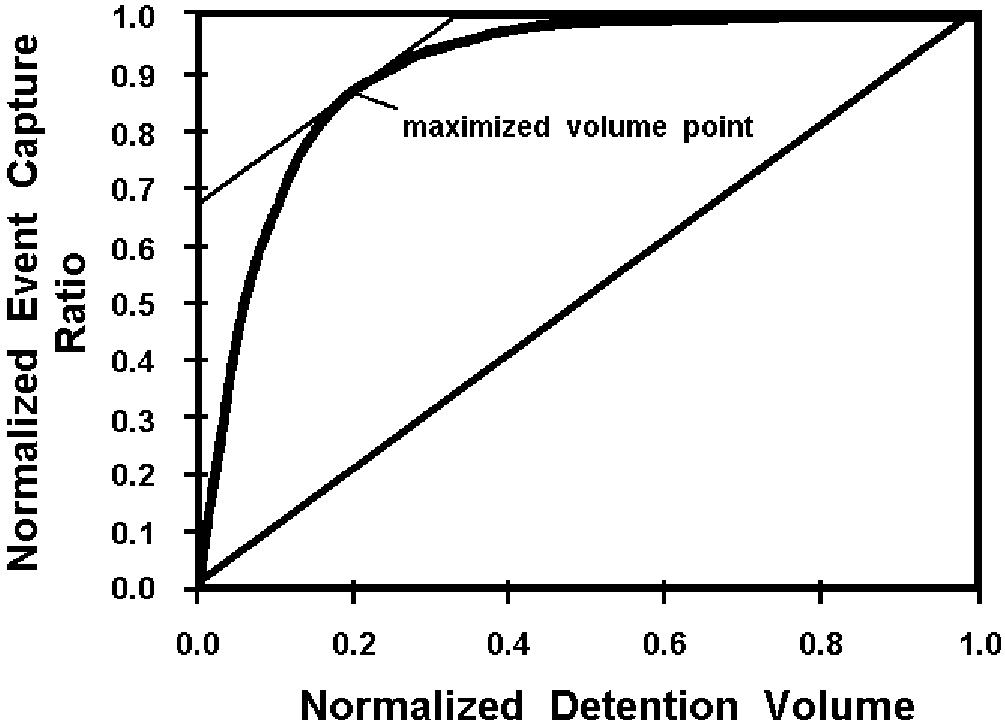


Figure 7-3. Ratio of events captured as a function of the normalized detention volume. (Urbonas et al*.,* 1990).

The finding of a maximized volume point at all rain gauge records studied throughout U.S. prompted Guo and Urbonas (1996) to search for a relationship between the mean runoff producing storm depths reported by Driscoll et al. (1989) and the maximized capture volume. Such a relationship was found in 1993, and was later simplified by Urbonas et al*.* (1996) into a simple linear function. WEF & ASCE (1998) adopted this relationship and recommends its use for simple on-site designs and initial planning efforts.

Grizzard et al*.* (1986), based on laboratory and field studies in the Chesapeake Bay area, suggested that detention basins need to capture the runoff from a mean storm and hold it for an extended period of time to effectively remove pollutants associated with total suspended solids (TSS). They suggested that such a detention basin be equipped with an outlet that released its full volume in 24 hours or more.

This concept was examined using continuous modeling to test the sensitivity of the capture volume size for the Denver area (Urbonas et al. 1990). Table 7-1 summarizes these findings and shows is that the idea of “bigger is better” is not justified for TSS removal by a retention pond equipped with an extended detention surcharge volume above its permanent pool. Field studies at the Shop Creek pond facility in Aurora, CO (Urbonas et al. 1993) produced results consistent with these findings.

The need to focus on TSS removal by BMPs has been recently reinforced by DiToro et al. (1993) and Cerco (1995). They both studied bottom sediment in receiving waters and found that sediment deposits in Chesapeake Bay can have a benthic oxygen uptake. Thus, TSS reduction in stormwater runoff can be the primary reason for selecting and sizing many structural BMPs.

Table 7-1. Sensitivity of the BMP capture volume in Denver, CO (Urbonas et al. 1990).

Capture Volume to Maximized Volume Ratio	Percent of Annual Runoff Volume Captured	Percent of Average Annual TSS Removed
0.7	75	86
1.0	85	88
2.0	94	90

Thus, in order to be effective in the removal of most constituents found in stormwater, structural BMPs need to focus on the frequently occurring smaller events. As a result, detention and retention facilities, wetlands, infiltration facilities, media filters, water quality inlets, swirl concentrators and possibly swales need to be designed to accommodate the runoff volumes and flow rates that result from smaller storm events. It has been recommended that the capture volume for water quality enhancement and for the protection of receiving stream integrity be somewhere between the runoff volume from a mean storm event (Driscoll et al.,1989) and the maximized volume (Urbonas et al. 1990, Hall et al.,1993, Guo and Urbonas 1996). Furthermore, this volume should be released over an extended period of time, namely, somewhere between 12 to 48 hours (Grizzard et al. 1986, Urbonas et al., 1990, Urbonas and Stahre 1993).

Other design considerations, however, come into play when dealing with the removal of nutrients and dissolved constituents. The permanent pool volume of ponds, the volume and the surface area of wetlands and other biochemical dependent BMPs (e.g., peat-sand mix filter) need to be designed and sized on considerations other than only capture volume (Hartigan 1989, Lakatos and Mcnemer 1987, Galli 1990). Nevertheless, even these facilities are likely to benefit from a surcharge capture volume sized as discussed in the preceding.

An Assessment of Best Management Practice Effectiveness

Non-Structural Best Management Practices

Non-structural BMPs rely on human behavioral changes to reduce the amount of pollutants that enter a separate stormwater system, which transports untreated stormwater and the pollutants it contains to receiving waters such as arroyos, gullies, brooks, streams, lakes, estuaries, and reservoirs. As a result, quantifying the amounts of various constituents (some of which may be pollutants) that non-structural practices eliminate from being delivered to these receiving waters is very difficult.

Some of these practices directly affect the types and numbers of structural BMPs that are going to be used as land development and redevelopment takes place. As a surrogate measure, the effectiveness of the structural controls, and the percentage of the total urban landscape within a community or a watershed these controls intercept, can be used to quantify the effectiveness of the regulatory, non-structural practices.

On the other hand, how does one measure the amount of pollutant load that does not reach the receiving systems because of educating the public or a change in behavior? USEPA (1993) goes into much discussion and detail on what to do and how to do it, but does not provide reliable methods for quantifying the effectiveness of non-structural BMPs in reducing pollutant loads reaching the receiving waters of this nation.

The discussion that follows attempts to address some of the issues and questions regarding non-structural BMP effectiveness. It draws on many discussions involving municipal public works and park department officials in Colorado and other states. Some of it interprets and adds to the issues discussed by USEPA (1993). Unfortunately, no field data is known to exist on the effectiveness of many of these practices on reducing the pollutant loads reaching receiving waters. However, several field studies are under way, the most prominent known study being the one in Portland, OR. Hopefully, with sufficient data from well controlled field investigations, some of the outstanding questions will begin to be answered.

Pollutant Source Controls

For this practice to be effective, widespread changes must occur in the use of various potentially polluting products. It is insufficient for a single city or metropolitan area to discontinue the use of a product it believes to pollute its waterways because such a product will be brought in from outside from adjacent communities where it is still being used. For example, requiring that only phosphorous free or low phosphorous detergents be sold will only work if such a ban is state or nation wide.

On the other hand, municipalities and industries can, through proper training and licensing, probably reduce the amount of certain types of pollutants applied to their landscapes. Through changes in the traditional ways some of these institutions handle and apply various materials to the urban landscape in their daily maintenance and operation activities, loads of various materials reaching the surface waters can probably be reduced. For example, proper application of pesticides and herbicides and minimizing their overspray will reduce the amount of these chemicals applied on the vegetated and adjacent impervious surfaces. Also, the calibration of equipment to minimize the rate of salt and other deicing chemicals being applied to road surfaces in winter should also reduce the loads of these chemicals reaching the receiving waters and groundwater when ice and snow melts. Other possible municipal practices that can help reduce pollutant loads reaching the receiving waters could include the licensing and training of pesticide and herbicide applicators; controls on how and where commercial carpet cleaners dispose of their waste water; building codes requiring rain covers over fueling pumps, mechanical maintenance areas, and chemical storage and loading areas; and proper storage and handling of garbage disposal bins at food handling institutions such as restaurants and other commercial and industrial activities.

Intuitively, all of these can reduce the amount of pollutants applied to the urban landscape. However, to what degree these practices actually reduce the amount of various pollutants reaching the receiving waters, or if the quantities being reduced actually make a difference to the water quality of the receiving waters, has yet to be quantified. If only insignificant gains in receiving water are in fact possible, are all or any of these practices remotely cost effective? These questions still need carefully designed field studies to answer. One question that remains is how aggressively should municipalities pursue such non-structural controls and practices before answers about their effectiveness are in. Should the municipalities focus primarily on practices they know work well for the site specific conditions of their community?

Public Education and Citizen Involvement Programs

The goal of public education according to those involved in the field is to modify behavior. That is also the stated goal of US EPA (1993). To be effective, modifications are needed in how a large majority of individuals use and dispose of fertilizers, pesticides, herbicides, crankcase oil, antifreeze, old paint, grass clippings and many other products that contain toxicants, nutrients or oxygen demanding substances. To what degree and in what numbers changes in behavior can be achieved through public education has yet to be answered.

The belief is that the more aggressive the education program, the more effective it should be. This has to be questioned, since there probably is a point of diminishing returns. Where that point is has yet to be determined and will probably be, to one degree or another, a function of the economic, social, ethnic, educational and language makeup of the population being targeted. For public education to work, the target public has to care, or has to be convinced to care. Simple distribution of information through mass media or through written materials is not likely to achieve widespread acceptance of the message or results in terms of water quality improvements.

Walesh (1993, 1997) advocates a proactive public involvement program that goes beyond public education, which tends to be one-way “communication,” and instead reaches for public involvement, which constitutes to two-way communication. Guiding principles of these public involvement programs include:

- A public interaction program, or lack thereof, is often the principal reason for the successful implementation of an urban water program or the failure to implement it.

- The success of a public involvement program is determined more by the total number of different “publics” that participate than by the told number of individuals involved.

- Essential to the success of a water management effort is agreement between the public and the water professionals on what problems are to be prevented or mitigated.

In addition to public education and involvement efforts, communities need to have programs in place that make it convenient for the public to dispose of unwanted household products and toxicants. Disposal centers with easy access need to be in place so the public can, in fact, follow through on what is being asked of them.

Street Sweeping, Leaf Pickup and Deicing Programs

Field tests by US EPA (1983) demonstrated that street sweeping reduced by very little the concentrations of constituents reaching receiving waters. It may be possible, however, that strategically scheduled sweepings at key periods of the year can reduce constituent loads available for wash off by stormwater. For example, in the midwest, sweeping in the fall and in late winter months can reduce the leaf litter and street deicing products reaching receiving waters. With current technology, street sweeping is most effective in picking up coarse sediment and litter, thus enhancing the aesthetics of stormwater discharges.

Local Government Rules and Regulations

Well drafted ordinances, rules, regulations and criteria and their enforcement can provide the basis for an effective stormwater management program especially in providing structural BMPs and erosion and sediment control for new land development and redevelopment. Such local ordinances, rules and regulations can help reduce impacts of urban runoff from newly urbanizing lands by providing for and/or requiring:

1. Installation of structural BMPs as land develops or redevelops. This is less expensive than retrofitting structural BMPs later.

2. Enforcement of site disturbance and erosion control programs.

3. Encouragement of the use of minimized DCIA in new development, including the use of landscaped areas, grass buffers, and roadside swales instead of curb, gutter and storm sewer whenever site conditions and land uses permit.

4. Maintenance for publicly owned BMPs.

5. Enforcement of the operation and maintenance of privately owned stormwater management facilities, including on-site structural BMPs and non-structural measures.

Elimination of Illicit Discharges

Untreated wastewater discharged through illicit connections is a public health concern, which justifies efforts to find and eliminate illicit wastewater connections. Illegal dumping, however, because to its covert nature, is extremely difficult to control and soliciting the help of the public to report suspicious or apparently illegal activities may be one way for extending its effectiveness.

Structural Best Management Practices: Design Considerations

Many factors influence the effectiveness of any structural stormwater BMP installation. Although progress has been in understanding how some of these controls perform, selecting, sizing, designing, operating and maintaining effective BMPs for the purpose they are intended to serve is still a challenge. Many BMPs are used without full understanding of their limitations and their effectiveness under field conditions, which often differs from regulatory expectations or modeled predictions. This is particularly the case when addressing the effects of urbanization on the receiving waters.

What is a particular BMP supposed to address? Is it the removal of suspended solids, or is it the removal of dissolved metals or is it the organic matter in the sediment that can settle on the bottom and cause sediment oxygen demand on the water column? Which of these or other “problems” is most important when selecting a single BMP or a group of BMPs? For instance, recent bottom sediment studies reveal that these sediments can have a significant benthic oxygen uptake and may be the cause of oxygen sags and suppression of micro invertebrate populations in the receiving waters (Cerco 1995, DiToro and Fitzpatric 1993). If that is the case, the removal of sediment may be the primary reason for selecting the BMP instead of nutrients that have also been linked to oxygen sags. Or should the selection of the BMP be driven by the need to reduce flow rates and volumes of runoff from urbanizing areas? These and other factors need to be considered in planning for maintenance and/or the restoration, or determining the inability to attain a desired restoration level, and recommending a family of BMPs for use in any given watershed.

Local Climate

As a first step, one needs to consider local climate. If the treatment control relies on a "wet" condition for vegetation and biological processes, the site needs adequate ambient precipitation throughout all seasons. In arid and semi-arid areas, such as the southwest, such treatment controls are not practical unless supplemental water is provided to make up for the evapotranspiration during dry seasons. Thus, when assessing the effectiveness of structural controls, the suitability of the practice for the local climate and meteorology must be considered.

Design Storm

The use of an appropriate design storm to size a facility is probably one of the most important considerations. Often some designers and regulators believe that the bigger the design storm the more effective the control facility will be. That often is far from the truth. Controls designed to improve stormwater quality and to control downstream flow rates need to be matched with the type of facility being used, local hydrology and the receiving system needs. Use of an appropriate design hydrology to design each control facility is assumed in developing the various assessments of BMPs that follow.

Nature of Pollutants

The nature of stormwater pollutants has to be considered when selecting and sizing BMPs. Most BMPs are suited for the reduction in suspended solids and of the dissolved fraction of constituents that attach to these particles. If, however, the removal of nutrients and dissolved constituents is the goal, the family of suitable BMPs is much smaller. The concentration of a constituent in the water column has an effect on the "efficiency" reported for the BMP. When high concentrations are present the BMP will typically show higher percentages of removal than when low inflow concentrations are encountered. For this reason, the reporting of effectiveness in terms of percent removed has to be questioned. This is evident when the water quality of the effluent is very good and the percent removal is low. This may be because the inflow concentration of the constituent of concern is also low.

Figure 7-4 compares the "efficiency of removal" in percent to the actual effluent concentrations for total phosphorous by a sand-peat filter as a function of influent concentration for one set of field tests. Tests for other constituents at this same site produce somewhat less definitive relationships, but a similar general trend was observed. Figure 7-4 is probably one of the more dramatic illustrations of the fact that the influent concentration affects the percent removal rate. It implies that a mathematical relationship can be developed for this site. It may even be possible to develop similar relationships for other BMPs and other sites, but that has yet to be demonstrated with sufficient variety of field data. Although a similar form for such an equation may possible, the regression coefficients are likely to differ for each constituent, each BMP type and, possibly, for each site. Nevertheless assuming such a relationship is possible, Figure 7-4 suggests a general form such as % Removed = $100*[1- (c/C_i)^k]$, in which c and k are regression constants and C_i is the influent concentration.

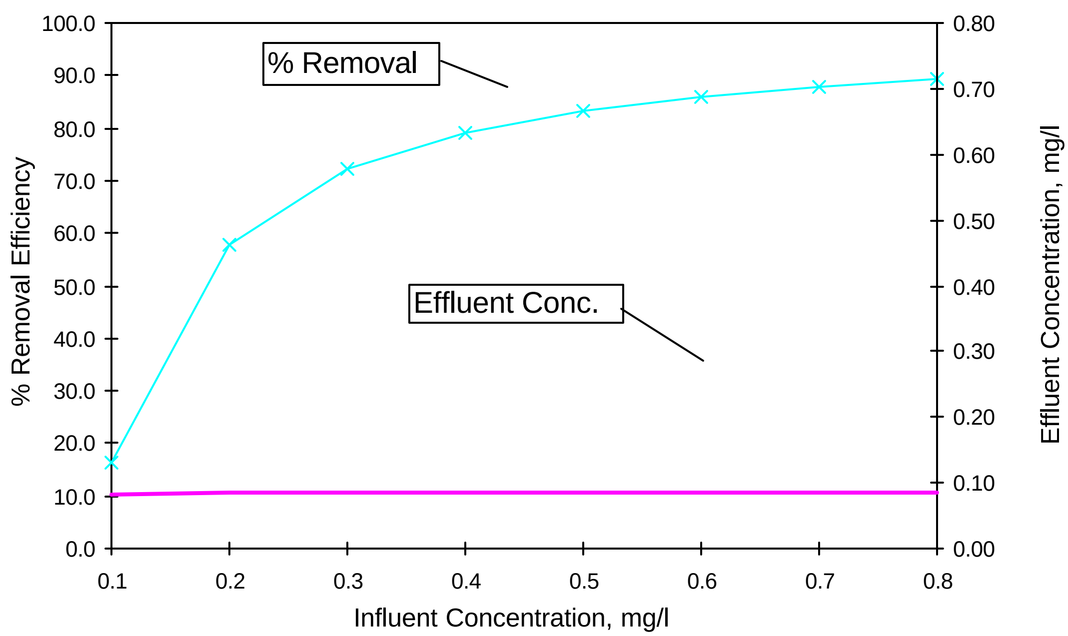


Figure 7-4. Total phosphorous "percent removal efficiency" and effluent concentrations for a peat-sand filter as a function of influent concentration. (Farnham and Noonan 1988).

Based on the preceding discussion, the definition of effectiveness should be based on more than “percent removal” of a constituent. It may be more appropriate to judge effectiveness against ranges of realistic effluent concentrations or some other parameter established by local watershed studies. It is not appropriate, however, to base this judgment on water quality standard developed for continuous dry weather flows, or on fixed percent removals of a constituent.

Often a community judges the “effectiveness” of a BMP by what other attributes it possesses, or what uses, other than stormwater management, it offers to the community. Thus, the incorporation of one or more other uses, namely multiple uses, such as active and passive recreation, enhancing or protecting wildlife habitat, flood control, and ground water recharge, into the BMPs design often is considered by the local residents as an “effective” facility. In contrast, a single-purpose, well functioning stormwater management facility sometimes is judged by its neighbors as a “nuisance.”

Operation and Maintenance

Operation and maintenance practices, or lack thereof, can significantly influence the actual effectiveness of structural BMPs. Most treatment controls do not require active operation of mechanical or chemical systems equipment, but all need adequate maintenance. Provision of such maintenance is assumed in the assessment discussions that follow. Also assumed in these discussions is that appropriate soil erosion controls are being vigorously practiced within the tributary catchment. If not, even the best designs can be rendered inoperative because of large sediment loads generated by uncontrolled construction sites.

On-Site or Regional Control

Another issue that needs to be considered is whether a BMP is used as an on-site or as a regional control. Very large numbers of on-site controls, sometimes exceeding several hundred or even several thousand, may be in place within any urban watershed. Reliably quantifying their cumulative hydrologic impacts on receiving waters becomes virtually impossible. Water quality, however, can be improved by both regional and on-site controls.

The degree of improvement for the cumulative effect in numerous on-site controls is, however, less predictable than with regional controls. This is because large numbers of on-site controls seriously complicate the quality assurance efforts during their design and construction. Large numbers of on-site controls are designed by a variety of individuals, which are then constructed by a variety of different contractors under varying degrees of quality control. Furthermore, very large numbers of BMPs will be maintained and operated in a variety of ways that are virtually impossible to anticipate or to effectively control.

Wiegand et al. (1989) estimated that regional controls are more cost effective because fewer controls are less expensive to build and to maintain than a large number of on-site controls. Regional controls can provide treatment for existing and new developments and can capture runoff from public streets, which is often missed by many of the on-site controls (Urbonas and Stahre 1993).

The major disadvantage of regional stormwater controls, such as detention basins, is that they require advanced watershed planning. Even when such a plan exists, the necessary up-front financing may be out of phase with the land development that is occurring in the watershed. Often the use of on-site controls is the only practical institutional, financial and political alternative.

Structural Best Management Practices: Performance

A number of the most commonly used structural BMPs are discussed next. Each is evaluated as to its effectiveness in addressing water quality, control of runoff volume and ability to moderate runoff rates in the receiving system. Also, when appropriate, some or all of the other points mentioned above are addressed.

Minimized Directly Connected Impervious Area

This practice has been around for a long time. However, up until recently it was recognized or defined as a stormwater management practice. In fact, it has been considered as inadequate and inappropriate for "good drainage" in urban areas. For certain types of urban land uses this practice can be a very effective stormwater BMP.

Unfortunately there are no data to show how much the implementation of minimized DCIA reduces surface runoff volumes, peaks and pollutant loads. The exact performance of this practice depends on which types of components show on Figure 7-2 are used at the site, the exact nature of the local geology, the type of soils and vegetative cover, and the nature of local climate. Under ideal conditions, surface stormwater runoff from low to medium density single family residential areas can be virtually eliminated for small rainstorms (i.e., storms with less than 13 to 25 mm (0.5 to 1.0 inch) of rainfall).

On the whole, this is a very effective stormwater BMP for low to medium density residential developments and for smaller commercial sites. Minimized DCIA is not a very effective BMP for high density residential developments and high density commercial zones, such as central business districts. This BMP demands that much of the land area of the development have a pervious surface, free of buildings and solid pavement. It may also not be appropriate for use when the general terrain grades are steeper that six percent. With highly erodeable soils, minimized DCIA may require even flatter terrain slopes.

This is one of the very few BMPs that, when used appropriately, can moderate the flow effects of urbanization in receiving waters, especially from the smaller storms. Also, for low to medium density developments, it can save on the cost of drainage systems and could be cost effective because the cost of storm drainage systems are reduced. In addition, with the use of stabilized shoulders, the surface area of pavement on public streets can be less than is used for a traditional street cross-section, thereby saving on initial construction and on its maintenance.

If misused, minimized DCIA can result in many problems to local residents that are often the result of poor drainage. Such problems include boggy mosquito breeding areas, poor snow removal and hazardous roadside ditches. On steeper slopes, erosion along some roadside and backyard swales has been observed. Also, property owners have been observed paving and filling poor draining, eroding or deep swales fronting their yards. Local policing and enforced preservation of the swales may be needed to prevent their loss through actions of local residents. Such enforcement is not a politically popular prospect for locally elected officials, especially if the citizens believe they are eliminating a problem on their front lawn.

This practice not be used for industrial and commercial sites that may be susceptible to spillage of soluble pollutants such as gasoline, oils, or solvents. The concern is prevention of soil and groundwater contamination.

Grass Swales

Removal rates exceeding 80% of TSS by grass swales are suggested by Whallen and Cullum (1988). Others suggest lower removal rates, on the order of 20 to 40% (UD&FCD 1992). The higher rates suggested by Whallen and Cullum may be possible when soils have very high infiltration rates and very slow flow velocities occur (i.e., less than 0.15 m/s). Grass swales appear to be best suited when terrain slopes are less than 3% to 4%, although some have suggested their use with terrain slopes as high as 6%. The limitations of site overlot grading during land development make the effective use of swales at higher slopes not practical. The use of swales is an integral part of the minimized DCIA practice.

The use of grass swales as stormwater collectors, instead of curb-and-gutter, slows the runoff process and can, under certain site conditions, also reduce the volume of runoff. Unless the swale is underlain by a clay layer, it is not recommended for use at industrial and commercial sites that may be susceptible to spillage of soluble pollutants such as gasoline, oils, and solvents for fear of soil and groundwater contamination.

Grass Buffer Strips

Grass buffer strips can remove larger particulates and promote local infiltration, provided the flow is kept very shallow and slow. Under ideal conditions, removals of 10 to 20% of suspended solids have been suggested (UD&FCD 1992). Buffer strips are an integral part of the minimized DCIA practice and are also an important part, of a number of practices that act in combination with each other. Thus the use of grass buffer strips is suggested whenever site conditions and land uses permit, upstream of swales, infiltration, percolation, wetlands, retention, and detention type of BMPs.

The use of grass buffer strips can slow surface runoff and, under certain site conditions, also reduce the volume of runoff, especially from small storms. Unless the grass buffer strip is underlain by a clay layer, it is recommended that it not be used at industrial and commercial sites that may be susceptible to spillage of soluble pollutants such as gasoline, oils, and solvents for fear of soil and groundwater contamination.

Porous Pavement

Field evidence indicates that properly designed modular pavement block porous pavement may be the only form of porous pavement that has a proven long-term successful performance record. This type of pavement has been in use since the mid-1970's with very few reported problems (Day et al. 1981, Smith 1984, and Pratt 1990). When porous pavement begins to clog, the simple removal and replacement of the soil or sand media in the pavement's openings can return it to full function.

On the other hand, Schueler et al. (1991) and others have reported that monolithic porous pavement surfaces tends to seal within one or two years after their installation. Once sealed, return the pavement to an acceptable working level is virtually impossible without total replacement of the pavement. Estimates of constituent removals for modular porous pavement range from 65 to 95%, depending on the constituent being monitored and the nature of local site and meteorological conditions.

The use of porous pavement can slow surface runoff and, under certain site conditions, reduce the volume of runoff, especially from the smaller storms. Unless porous pavement is underlain by an impermeable membrane and the stormwater is collected by an underdrain for surface discharge or post-treatment, the use of porous pavement not be considered for industrial and commercial sites that may be susceptible to spillage of soluble pollutants such as gasoline, oils, and solvents, for fear of soil and groundwater contamination.

Percolation Trenches

When properly operating, percolation trenches can remove up to 98% of the suspended solids in the stormwater and many of the constituents that are associated with these particulates. It has also been asserted that these facilities can also remove significant faction of nutrients, metals and other constituents from surface runoff. However, there is a concern that groundwater contamination may occur.

When operating, percolation trenches can reduce the volume of stormwater surface runoff. In fact, they can virtually eliminate direct surface runoff from small storms (i.e., less than 13 to 25 mm (0.5 to 1.0 inches) of precipitation).

Schueler et al. (1991) report that about 50% of percolation trenches constructed in the eastern U.S. have failed. He did not report on the nature and reason of these failures, although clogging within the trench and of its infiltrating surfaces were suspected. Two comprehensive field inspections, one in 1986 and the other in 1990, of percolation trenches were performed by the State of Maryland (Pensyl and Clement 1987, Lindsey et al.,1991). During the 1990 inspection of 88 percolation trenches, 51% showed signs of partial or major failure. Also reported was the fact that 31% of those failures occurred between 1986 and 1990. Although only 45% of installations reported a need for sediment removal maintenance, the inspectors reported a high incidence of sediment entering these trenches. Discussions with stormwater professionals working in the eastern U.S. indicates that the failure rate may actually be higher in 1996 than was originally reported by Schueler et al. (1991) and Lindsey et al. (1991).

It is possible to postulate from the inspectors' descriptions that clogging of percolation trench surfaces and groundwater mounding are the two most likely contributors to the reported failures. Groundwater mounding can develop under and around a percolation trench, actually surfacing within the trench (Stahre and Urbonas 1990, Colorado Storm Water Task Force 1990).

Clearly, the use of this practice should not be encouraged until sound engineering design guidance is adopted, possibly similar to the methodology suggested by Urbonas and Stahre (1993), including pre-filtration of stormwater before it enters a trench and the use of a comprehensive groundwater hydrologic investigation during design. Furthermore, percolation trenches should not be used at industrial and commercial sites that may be susceptible to spillage of soluble pollutants such as gasoline, oils, and solvents for fear of soil and groundwater contamination.

Infiltration Basins

Properly operating infiltration basins can remove anywhere from zero to as high as 70 to 98% of the pollutants found in stormwater, depending on the constituent and site conditions. Also, when operating, infiltration basins can reduce the volume of stormwater runoff and virtually eliminate direct surface runoff for small storms (i.e., less than 0.25 to 0.5 inches of precipitation).

Two comprehensive field inspections, one in 1986 and the other in 1990, of infiltration basins were performed by the State of Maryland (Pensyl and Clement 1987, Lindsey et al. 1991). During the 1990 inspection, 73% of the 48 installations inspected were judged as "failed." The inspectors reported that only 41% of the inspected infiltration basins needed sediment removal maintenance. From the inspectors' descriptions, groundwater mounding appears to have contributed to some of the reported failures. Their rate of failure implies a lack of sound engineering in their design and/or construction. Lack of maintenance may have contributed to some of the reported failures, but the findings by Lindsey et al (1991) suggest that other factors were at work in many of the reported failures.

This practice should not be encouraged until sound engineering design guidance is adopted, possibly similar to the methodology suggested by Urbonas and Stahre (1993). When operating properly, infiltration basins can reduce the volume of stormwater surface runoff. In fact, they can virtually eliminate direct surface runoff from small storms (i.e., less than 13 to 25 mm (0.5 to 1.0 inches) of precipitation).

Infiltration basins not be used for industrial and commercial sites that may be susceptible to spillage of soluble pollutants such as gasoline, oils, and solvents for fear of soil and groundwater contamination.

Media Filter Basins and Filter Inlets

Filters can be very effective BMPs where land area is at a premium, but they need regular maintenance. When they are undersized or are left unmaintained, media filters accumulate a layer of fine sediment on their surface and seal. Once clogged, a media filter drains at very slow rate and stormwater runoff either ponds upstream of the filter or bypass it (Urbonas et al. 1996b). Either condition is unacceptable. In the first case the ponding water may be a nuisance or create dangerous situations. In the latter, only a fraction of the stormwater that arrives at the filter actually receives the treatment efficiencies typically reported for sand filters.

To compensate for this potential problem, oversizing the filters or providing stormwater capture detention volume upstream that is sized in balance with the filter's clogged flow-through rate is necessary. Both approaches, that is, oversizing and upstream detention, might be used. Oversizing the filter can also reduce the necessary frequency of maintenance. Providing an extended detention basin for pretreatment is suggested by Urbonas and Ruzzo (1986) and Chang et al. (1990). Field experience with designs that have a full presettlement detention basin appear to have much longer life before the filter surface requires cleaning and/or the media needs replacement.

Tests using media other than sand, such as peat, peat-sand mix, compost-sand mix show them to clog faster than sand filters (Galli 1990, Stewart 1989). This means their longevity at acceptable hydraulic flow through rates may be very poor and they may be even less attractive and functional than filters using sand as the media for filtration.

When a media filter is located within an underground vault, such as a water quality inlet, it is out-of-sight-and-out-of-mind and is likely to not receive the needed maintenance attention of a visible surface facility. Regular inspection programs are a must if media filters are used in order to assure their continued proper operation.

A media filter basin or inlet, without an upstream detention basin, has no effect on stormwater runoff flow rates. As a result, these facilities have no potential for attenuating increases of runoff rates from urban areas.

Sand filter inlets suggested by Shaver and Baldwin (1991), while effective, are expensive to construct. Above ground filter basins are also significantly more expensive to build than detention basins. It has been argued that media filters are most likely to be used where land costs are very high. However, comparisons of filters, designed with clogging and minimal maintenance in mind, to detention basins and retention ponds revealed that the filters require similar land areas to construct as do detention basins. If this is the case, as recent findings have suggested (Urbonas et al. 1996 b), the cost of functional media filters may actually be more than detention basins. Also, based on the analysis of various unit operations and filter clogging processes measured under laboratory and field conditions, Urbonas (1997) suggested an engineering design and analysis procedure for stormwater runoff sand filters. This procedure provides for design and water quality performance by accounting for runoff probabilities, suspended sediment loads in stormwater, volumes processed by the filter and volumes bypassing it and the maintenance (i.e., cleaning) for the filter media.

Water Quality Inlets

Episodic evidence reported by a number of observers over a number of years and more recently confirmed by Schueler et al. (1991) through field tests, indicates poor performance by water quality inlets (i.e., sand and oil and grease traps). These devices, depending on their complexity, can be very expensive to construct and to maintain and appear to offer very little water quality enhancement in return. Also, these devices provide no peak flow or volume control capability. Additional, research and development efforts are likely to occur in this area.

Swirl-Type Concentrators

Swirl concentrators are designed to process stormwater up to a stated design flow rate and to by-pass flows that exceed this rate. When they work properly, swirl concentrators can remove the heavier sediment particles and many of the floatables found in stormwater. They have not been shown to be effective in the removal of neutrally buoyant solids such as plastic bags, oils, greases or very small or light suspended particles. Also, they have been known to perform below expectations for larger and smaller flow rates than the specific design rate.

New commercial devices such as StormCeptor™ are currently being field tested and objective results on their performance should begin to show up in literature within the next two years. These devices can be expensive to construct and to maintain. Swirl concentrators provide no peak flow or volume control capability unless they have a detention basin upstream of them to equalize flows.

Extended Detention Basins

The performance of a relatively large number of extended detention basins have been documented by field and laboratory tests. For example, removal rates for TSS range from 10 to 90%, depending on the constituent being sampled, the geometry of the installation, and the local climate. For properly sized and designed extended detention basins, removal rates for TSS, lead and other undissolved constituents are only somewhat less than observed for retention ponds and wetlands. Although sedimentation is the main treatment process in these basins, other associated processes are known, or are suspected, to be at work. These include flocculation, agglomeration, ion exchange, adsorption, physical resuspension of particulates, and solution.

According to Grizzard et al. (1986), to serve as a water quality enhancing BMP, a detention basin needs to hold stormwater runoff for much longer periods of time than a detention basin that is used for the purpose of controlling peak runoff rates. Thus the term extended detention basin has been coined. For the smaller storms, namely the storms that produce somewhere between the mean and the 90th percentile surface runoff volumes, the minimum emptying time of the captured volume needs to be between 24 to 48 hours (Grizzard et al. 1986, Urbonas et al. 1990, Urbonas and Stahre 1993). To be most effective for water quality enhancement and to mitigate some of the effects of increased surface runoff from an urbanizing area, the longer of the suggested drain times needs to be used with the larger design storm (i.e., probably exceeding 13 to 20 mm [0.5 to 0.75 inches] of precipitation) and the shorter drain times with the smaller events (i.e., probably less than 13 mm [0.5 inches] of precipitation).

Extended detention basins can be designed to control the flow rates from a wide range of small to large storm runoff events. However, the most difficult storm events to control are the small ones from small tributary areas. The outlet needed to throttle flows down to very low levels needs to have very small openings, which are susceptible to clogging. Control of the larger events is accomplished by the detention volumes that surcharge the water quality extended detention volume. Also, an extended detention basin does not reduce the volume of the runoff that enters it.

Retention Ponds

Hartigan (1989) stated that retention ponds can remove 40%-60% of phosphorus and 30%-40% of total nitrogen. Other studies show lesser annual removal rates. Studies in Washington, DC area by Schueler and Galli (1992), indicate that the permanent pools characteristic of retention ponds can act as heat sinks resulting in warm water releases and, therefore, retention ponds may not be appropriate for use if they discharge to streams that support trout. Often a retention pond is sized to remove nutrients and dissolved constituents, while any pool that may be associated with an extended detention basin is much smaller and is provided for aesthetics, namely, to cover the solids settling areas with water.

The major features of a state-of-the-art design of a retention pond includes a permanent pool and an emergent wetland vegetation bench called the littoral zone. The pond provides a volume of water where the solids can settle out during the storm event (i.e., active sedimentation period) and during the periods between storms (i.e., quiescent sedimentation period). Sedimentation can also remove that fraction of nutrients and soluble pollutants that adhere to sediment particles. The littoral zone provides aquatic habitat, enhances the removal of dissolved constituents through biochemical processes and helps to minimize the formation of algae mats. Sometimes the pond has surcharge detention storage volume above it that can be used for flood control and to enhance sedimentation during storm runoff periods.

Retention ponds, on the average, can do a noticeably better job at the removal of nutrients than extended detention basins. However, the reported variability in performance ranges for retention ponds indicate that much remains to be learned about their performance. This knowledge will be needed to develop a reliable design guidance for nutrient removals. Nevertheless, the use of retention ponds appears to be more effective than extended detention basins, filters, swirl concentrators, swales, buffer strips, and other BMPs. A possible exception is constructed wetlands when nutrient loading is of concern, namely for urban watersheds that are tributary to reservoirs and lakes and to tidal embayments and estuaries.

For retention ponds to be effective in the removal of nutrients, the permanent pool has to have two to seven times more volume than an extended detention basin (Hartigan 1989), depending on local meteorology and site conditions. As a result, more land area is needed than is required for a detention basin and costs can be 50% to 150% higher than for an extended detention. This increase may not be as significant if the pond has surcharge storage for drainage or flood control peak-shaving.

Retention ponds can be more aesthetic than extended detention basins because sediment and debris accumulate within the permanent pool and are out-of-sight. Large retention basins are sometimes used as property value amenities, sometimes permitting surcharge in the “lake front” property cost. However, if the tributary area does not have sufficient runoff during the year, detention ponds can dry out or become unsightly “bogs” and become a nuisance to the adjacent property owners.

Thus, some of the issues to consider when choosing a retention pond are:

1. Can the tributary catchment sustain a sufficient base flow to maintain a permanent pool?

2. Are the receiving waters immediately downstream particularly sensitive to increased effluent water temperatures that can result from sun’s warming of the pond?

3. Do existing wetlands at the site restrict the design of the permanent pool of the pond?

4. Are water rights available for the evapotranspiration losses in states with a prior appropriation water rights laws?

Retention ponds can be designed to control the flow rates from a wide range of small to large storm runoff events. As with extended detention basins, the most difficult storm runoff events to control are the small ones, especially the ones from small tributary catchments. The outlet needed to throttle flows down to very low levels needs to have very small openings, which are susceptible to clogging. Control of the larger events is accomplished by the detention volumes that surcharge above the permanent pool. However, a retention pond does not appreciably reduce the volume of the runoff that enters it.

Wetlands

Properly designed and operated wetlands, on the average, can remove significant percentages of total phosphorous, nitrogen, TSS and other constituents from urban stormwater runoff (Strecker et al. 1990). However, when compared statistically to other BMPs, wetlands appear to remove most of the constituents found in stormwater to about the same percentages that one can expect from extended detention basins and retention ponds. The claim that wetland basins are more effective in the removal of nutrients from stormwater is probably true for some installations, while other installations appear to be less effective.

The ranges in the performance data reported for wetland basins tell us that much has to be learned about how wetlands function and what constitutes a reliable design, especially for nutrient removals. Well controlled field investigations are needed to identify which field conditions and design parameters produce consistently good pollutant removals.

For example, Walesh (1986) describes the planning and design of a restored wetland in series with a sedimentation pond intended to substantially reduce the transport of suspeded solids and phosphorous into an urban lake. Oberts et al. (1989) presents the results of a 29 month monitoring study of the system during which 19 rainfall and four snowmelt events were monitored. Total phosphorous removals were at or above 50% for rainfall events. The sedimentation pond-wetland system removed 90% the total suspended solids for all monitored rainfall and snowmelt events. The successful performance of the system, which, incidentally, exceeded the performance of four other similar systems in the area, was attributed to several factors. For example, pre-settling of stormwater runoff in the sedimentation pond prior to discharge into the restored wetland is important. The volume of the permanent storage pool should be at least 2.5 times the runoff volume generated from the mean summer storm. The area of the permanent pool in the sedimentation basin should be about two percent of the impervious area of the watershed and the pool should have the maximum depth of over four feet.

There are little data in literature on the performance of wetland channels. As a result, current estimates of their effectiveness are speculation and educated guesses. Extrapolations from limited data (Urbonas et al. 1993) suggest that properly sized and designed wetland channels compare well with the performance of wetland basins for nutrient removal during small storm runoff events and during dry weather flow periods.

Another claim found in the literature is that the removal of nutrients by wetlands requires regular harvesting of wetland basins. This claim, however, does not appear to be well substantiated by field data. In fact, the limited information that is available shows regular harvesting to be of questionable value in increasing nutrient removal rates. Mechanisms in addition to plant uptake appear to be responsible for nutrient uptake in nutrient removals by wetlands.

The actual mechanisms for the removal of phosphorous and of nitrogen by wetlands are probably different. Phosphorous removals are most likely associated with the removal of solids, including ionic adhesion to solids and uptake of the dissolved fractions by algae (i.e., eutrophication). When algae die, they are deposited on the bottom "muck" or benthos, taking along some of the phosphorus with them. However, these benthal deposits can release phosphorous under reducing conditions. Much of the phosphorous in the benthos, however, becomes permanently trapped and unavailable for release to the water column. Thus, the removal of the accumulated benthos (i.e., mucking out) has to take place occasionally to keep wetland basins and wetland channels operating satisfactorily.

Although the removal of nitrogen is, in part, the byproduct of algae and other plant uptake, nitrites and nitrates appear to be too mobile for effective removal rates by this process alone. Aerobic and anaerobic denitrification appears to also take place within wetlands. This process takes place in wetlands used for the polishing of wastewater treatment plant effluent, mostly in the root zones and on the biological film that is found on all wetland plants and their roots. Much of the current wetland treatment technology was developed for the treatment of wastewater (Nichols 1983, Kedlec and Hammer 1980) and has not had the benefit of the development for use under the vastly different conditions that occur during wet weather conditions. However, even for the uniform flow and loading conditions of a wastewater treatment plant, wetlands have a limit in how much nutrient loading they can accumulate before degradation in performance is experienced (Watson, et al. 1989). Much has yet to be learned about the actual bio-chemical processes at work in wetlands, especially for the treatment of stormwater, before it is possible to design them with confidence for stormwater treatment.

A wetland basin can be designed to control the flow rates from a substantial portion of small storm runoff events and to also control the flow rates from most large storm runoff events. The approach is to design them for the flow control function like one would design a retention pond.

Wetland channels can help control the flow rates of the smaller runoff events, however to a lesser degree than a wetland basin, an extended detention basin or a retention pond. Wetland technology is emerging as a viable tool for stormwater management but suffers from lack of prolonged field studies. Such studies are needed to answer questions such as how different wetland design configurations respond to stormwater loadings over an extended number of years when operating in the wide variety of climates, geologic settings and meteorological conditions found in the U.S.

Summary on Best Management Practice Effectiveness

Non-Structural Best Management Practices

A quantified assessment of how much effect non-structural BMPs have on the receiving water quality or the enhancement of its aquatic life has yet to be made. So far many surrogate measures have been used in an attempt to quantify their effectiveness. For example, the measure of gallons of oil recycled has been used to demonstrate how “effective” this non-structural BMP is, but this does not in any way quantify the number of gallons of oil this program eliminates from being transported to the receiving waters by the stormwater system. In other words, a surrogate measure may or may not have any relationship to the BMP’s effectiveness in reducing any specific pollutant from reaching the receiving waters or determining the impact on the receiving system.

Most of the suggested practices are supported by good intentions. For the most part they are a collection of common sense practices and measures. This leads to the belief that non-structural BMPs should provide a positive benefit when implemented and used, but data are needed to quantify the costs and benefits. If nothing else, non-structural BMPs should result in a cleaner looking urban landscape.

Structural Best Management Practices

The Definition of Effectiveness

Much more field performance data are available for structural than for non-structural BMPs. Table 7-2 summarizes the removal “efficiencies” of several structural BMPs most frequently used in the U.S. The table includes the information found through extensive literature reviews conducted for this report and by a Colorado task force (Colorado Storm Water Task Force 1990) and the Denver, Co area Urban Drainage and Flood Control District (UD&FCD 1992). What is of note are the wide ranges in the reported percent removals. Despite that, when properly designed for local soil, groundwater, climate and site geology, all BMPs will remove pollutants from stormwater to some degree. What is in question is how much at any given site and for how long will the BMP continue to function at those performance levels.

Table 7-2. BMP pollutant removal ranges in percent. (Bell et al. 1996, Colorado Storm Water Task Force, 1990, Harper & Herr 1992, Lakatos & McNemer 1987, Schueler 1987, Southwest 1995, Strecker et al. 1990, UD&FCD 1992, USGS 1986, US EPA 1983, Veenhuis et al. 1989, Whipple & Hunter 1981).

Type of Practice	TSS	Total P	Total N	Zinc	Lead	BOD	Bacteria
Porous Pavement	80-95	65	75-85	98	80	80	n/a
Grass Buffer Strip	10-20	0-10	0-10	0-10	n/a	n/a	n/a
Grass Lined Swale	20-40	0-15	0-15	0-20	n/a	n/a	n/a
Infiltration Basin	0-98	0-75	0-70	0-99	0-99	0-90	75-98
Percolation Trench	98	65-75	60-70	95-98	n/a	90	98
Retention Pond	91	0-79	0-80	0-71	9-95	0-69	n/a
Extended Detention	50-70	10-20	10-20	30-60	75-90	n/a	50-90
Wetland Basin	40-94	(-4)-90	21	(-29)-82	27-94	18	n/a
Sand Filters (fraction flowing through filter)	14-96	5-92	(-129)-84	10-98	60-80	60-80	n/a

Note: The above-reported removal rates represent a variety of site conditions and influent-effluent concentration ranges. Use of the averages of these rates for any of the reported constituents as design objectives for expected BMP performance or for its permit effluent conditions is not appropriate. Influent concentrations, local climate, geology, meteorology and site-specific design details and storm event-specific runoff conditions affect the performance of all BMPs.

The current definition of “effectiveness” in terms of percent removal is flawed, whether it is defined as the reduction in concentration or as the load of a constituent removed from stormwater runoff. A better measure needs to be developed to define how well a specific structural BMP is performing. This point was illustrated earlier by the example for the removal of phosphorous by a sand-peat filter. That example showed that the “percent removal” increased with the concentration of phosphorous in the influent while the concentrations in the effluent remained constant. As a result, “worst” performance was attributed to the storm runoff that had the cleanest water entering the filter.

Ironically, one can argue that a performance standard based on percent removals would be met most frequently when the watershed was kept in the most unclean condition, while the watershed with the best use of source controls would produce the worst performance record for the filter. This, despite the fact the filter's effluent was identical for both.

The nature of a redefined performance measure has yet to be determined. Such a standard will most likely be tailored for each structural BMP. It will have to address more than one question since the purpose for the selection and use of each BMP will vary with the local goals and objectives. As an example, is the BMP needed primarily to remove floating trash and sediment or is the removal of phosphorous or nitrogen the main goal, or is it the mitigation of increased runoff rates or volumes the main reason for the selection of the BMP? These and other, yet to be identified questions and issues will need to be addressed when developing a new "effectiveness" matrix for each BMP and its design.

Research and Design Technology Development Needs

While much is known about the performance of some of the discussed BMPs, such as retention ponds and extended detention basins, much more must be learned. For some BMPs, insight into their pollutant removal mechanism and characteristics is just beginning. For some areas of the U.S. there may even be sufficient information to relate BMP performance to a set of design parameters such as the size and imperviousness of the tributary watershed. This does not deny the fact that all BMPs can still benefit from well conceived and well controlled prolonged field studies.

An approach towards a systematic approach for performing field evaluations of BMPs was suggested by Urbonas (1995). Although there appears to be a significant number of BMP tests in the U.S. and other countries, what is lacking is a consistent scientific approach and the reporting of key design and tributary watershed parameters for the BMPs being tested. As a result of the data acquisition approach suggested by Urbonas, the American Society of Civil Engineers and the USEPA in 1996 entered into a cooperative agreement to define the data and information needs for such studies, to develop a data base software package for field investigators to use, to find and extract existing data on BMP performance, and to complete an initial evaluation of such data by the end of 1999.

To have significance, and to identify issues that arise over the near term, field investigations of BMPs probably need at least five years of data gathering, otherwise important performance information is likely to be missed. For some BMPs, performance is affected by maintenance and/or operations. For others, the maintenance needs will not become apparent for several years and prolonged testing is the only way to answer the question of how their performance will vary over time. Yet for other BMPs, performance may change over time. Such information will be needed to decide if and when such BMPs will need to be replaced or rehabilitated. Only when such information and much field performance data are available, are fully analyzed, and reliable relationships between performance and design parameters are quantified, will practitioners be in a position to design BMPs with performance expectations in mind. At this point there are too many unanswered questions on how to design BMPs for a stated performance level, whatever it may yet turn out to be. Among the questions that need to be answered are what kind of operations and maintenance are needed to provide the desired level of performance, what are the life cycle costs, and will they provide the desired results in the receiving waters for which they were selected or minimize the impacts of urbanization on those receiving waters?

Design Robustness

Robustness of BMP design technology is a factor that integrates what is known today about design. Robustness needs to be recognized when judging various BMPs for use. High robustness of design technology implies that, when all of the design parameters are correctly defined and quantified, the design has a high probability of performing as intended. In other words, the design technology is well established and has undergone the test of time. Low robustness implies that there are many uncertainties in how the design will perform over time. All facilities are assumed to be properly operated and maintained when judging design robustness.

Table 7-3 is an edited version of the collective opinion of many senior professional engineers involved in the development of the 1998 WEF & ASCE manual of practice for the selection and design of stormwater quality controls. The differences between this table and Table 5.6 of the MOP are based on further evaluation of the issues considered during the assessments at the time the MOP was being prepared. The weakest design link actually governs the overall design robustness of each BMP.

Runoff Impacts Mitigation

The emerging theme in the environmental community is the need for stormwater surface runoff flow control in urban and urbanizing areas. This concept has a long history of study and discussion in stormwater engineering literature. Changes in surface runoff hydrology with urbanization have been discussed by the engineering community now for over 20 years (McCuen 1974, Hardt and Burges 1976, Urbonas 1979, Glidden 1981, Urbonas 1983, Walesh 1989). The challenge until now has been to control the peak runoff rates for drainage and flood control purposes. This focus led to the control of peaks from larger storms such as the 5-, 10- or/and the 100-year flow rates. Use of on-site and regional detention became popular in some areas of the U.S.

Table 7-3. An assessment of design robustness technology for BMPs[1].

BMP Type	Hydraulic Design	Removal of Constituents in Stormwater		Overall Design Robustness
		TSS/Solids	Dissolved	
Swale	High	Low-Moderate	None-Low	Low
Buffer (filter) strip (2)	Low-Moderate	Low-Moderate	None-Low	Low
Infiltration basin (2)	Low-High	High	Moderate-High	Low-Moderate
Percolation trench	Low-Moderate	High	Moderate-High	Low-Moderate
Extended detention (dry)	High	Moderate-High	None-Low	Moderate-High
Retention pond (wet)	High	High	Low-Moderate	Moderate-High
Wetland	Moderate-High	Moderate-High	Low-Moderate	Moderate
Media filter	Low-Moderate	Moderate-High	None-Low	Low-Moderate
Oil separator	Low-Moderate	Low	None-Low	Low
Catch basin inserts	Uncertain	n/a	n/a	n/a
Monolithic porous pavement (2)	Low-Moderate	Moderate-High	Low-High (3)	Low
Modular porous pavement (2)	Moderate-High	Moderate-High	Low-High (3)	Low-Moderate

Notes:
1) Weakest design aspect, hydraulic or constituent removal, governs overall design robustness.
2) Robustness is site-specific and very much maintenance dependent.
3) Low-to-moderate whenever designed with an underdrain and not intended for infiltration.
4) Moderate-to-high when site conditions permit infiltration.

and Canada. In the early 1970s the State of Maryland was the first to require the control of the two-year peak flow rate for the stated purpose of controlling stream widening and erosion that were observed to take place after urbanization. However, Maryland acknowledges that the success of these requirements was well below expectations.

What is clear is that scientifically untested policies have little chance of success, despite their good intentions. They can lead to waste of resources and provide little or no environmental benefit, especially when applied through regulatory mandates. A better approach would be to develop long term field test beds before nationwide requirements or guidance on runoff flow controls are promulgated. Too much variety in community needs, ecological integrity protection, fiscal resources, physical settings of the receiving waters, climates, and geology exist throughout the U.S. to suggest a generic methodology. These type of decisions best rest at the specific watershed level and the state in which it is located.

The current demand by some for runoff flow controls has to be approached very carefully, lest resource (primarily in the form of land area and urban sprawl) consumption occurs without the commensurate environmental return. It is also possible to set up policies that physically cannot be met, such as “no increase in surface runoff volume.” Although some sites, under certain rainfall regimes, may be able to meet this standard after urbanization, this is probably not a realistic expectation at all sites, at all times.

Some of the BMPs discussed here can provide peak runoff rate mitigation. Others can provide mitigation of surface runoff peak rates and of runoff volume increases. None can totally eliminate the effects of urbanization. The most promising candidates for mitigating peak flow rates are the ones that capture runoff volume and release it over an extended period of time. These include retention ponds with extended detention surcharge volume over their permanent pool, extended detention basins, wetland basins and any other BMP that captures and slowly releases surface runoff.

Runoff volume reduction is much more difficult to achieve. Some of the BMPs discussed here can do so whenever site conditions permit. Trying to use such BMPs for volume reduction proposed under unfavorable site conditions is not only unwise, it is a gross denial of reality and physical limitations of the practices and the site conditions. For instance, these practices have only a limited potential for volume reduction when the development site is very steep, or has very tight or highly erosive soils, or is located in a region that cannot support a healthy and robust vegetative ground cover. Nevertheless, each of the BMPs is rated in the next section for their potential ability to reduce surface runoff flow rates and volumes.

Summary of the Usability of the Evaluated BMPs

Table 7-4 was designed to consolidate the foregoing discussion. It contains ranking scores from 1 through 5, with 5 being the score for the highest positive aspect and (-5) indicating the highest negative aspect of each BMP. As an example, potential for failure is considered to be a negative aspect, while the potential for mitigating the increases in surface runoff volume is considered a positive aspect. The rankings are based not only on what is reported in the literature, but also are based on experience in stormwater management. Clearly, the scores are somewhat subjective and further discussion and study are needed.

At any rate, the composite average rating scores reveal a ranking that integrates all of the aspects discussed and considered so far. Note the groupings of the BMPs. All ratings were ranked from one through 16 and then were segregated into five groups,

Table 7-4. Summary assessment of structural BMP effectiveness potential.

Structural BMP Type	Water Quality Improvement	Flow Rate Control	Runoff Volume Reduction	O & M Needs (1)	Sensitivity To Site Conditions	Failure Potential	Applicability for Given Land Use			Design Technology Robustness		Potential for Thermal Increases	Potential for Groundwater Contamination	Average of All Ratings	Rank Order of Rating Averages	Groupings by Rankings
							Low to Medium Residential	High Density Residential Medium Density Comm'l	High Density Commercial Industrial	Hydrologic and Hydraulic	Water Quality					
Minimized DCIA (2)	4	5	5	-3	-4	-2	5	3	1	4	4	-1	-3	1.09	**1**	***1***
Extended Detention Basin	4	5	1	-2	-2	-2	4	4	3	4	4	-3	-2	0.97	**2**	***1***
Retention Pond (3)	5	5	1	-2	-3	-1	4	4	3	4	4	-4	-2	0.97	**3**	***1***
Wetland Basin (3)	5	5	2	-3	-4	-1	4	4	2	4	3	-3	-2	0.85	**4**	***1***
Porous Pavement: Modular w/ Underdrain	3	5	1	-4	-2	-2	1	5	5	4	3	-2	-2	0.70	**5**	***2***
Infiltration Basin (2)	4	5	5	-4	-5	-4	5	5	2	3	4	-1	-4	0.64	**6**	***2***
Wetland Channel (3)	3	3	2	-3	-3	-1	4	4	2	4	2	-2	-2	0.58	**7**	***2***
Porous Pavement: Modular w/ Infiltration (2)	4	5	4	-4	-5	-4	4	5	5	4	4	-2	-4	0.61	**8**	***3***
Media Filter	4	1	0	-5	-1	-3	1	3	5	3	4	-2	-1	0.27	**9**	***3***
Percolation Trench (2)	4	4	4	-5	-5	-5	2	3	4	3	4	-1	-5	0.09	**10**	***4***
Grass Swale (2)	2	3	1	-3	-3	-2	5	3	1	3	1	-2	-2	0.09	**11**	***4***
Grass Buffer Strip (Grass Filter Strip) (3)	2	2	2	-3	-3	-2	5	3	1	2	1	-1	-2	0.09	**12**	***4***
Swirl-type Concentrator	3	1	0	-5	-1	-2	1	2	4	3	2	-2	-1	0.03	**13**	***4***
Dry Well (2)	4	4	4	-5	-4	-5	2	3	4	2	2	-1	-5	-0.09	**14**	***5***
Porous Pavement: Monolithic (2)	4	3	4	-5	-4	-5	3	3	3	2	3	-3	-4	-0.18	**15**	***5***
Water Quality Inlet	1	0	0	-5	-1	-3	1	2	3	3	1	-1	-1	-0.36	**16**	***5***

(1) Routine or rehabilitative maintenance, or both. **(2)** When site conditions permit. **(3)** When local climate site conditions permit

four with positive average ratings and one with negative ratings. The BMPs with the best average ratings were put into Group 1 and those with the lowest ratings into Group 5. These five groupings are as follows:

Group 1:	Minimized Directly Connected Impervious Area Extended Detention Basin Retention Pond Wetland Basin
Group 2:	Modular Porous Pavement With an Underdrain Infiltration Basin Wetland Channels
Group 3:	Modular Porous Pavement With Infiltration Media Filter
Group 4:	Percolation Trench Grass Swale Grass Buffer (Filter) Strip Swirl Concentrator
Group 5:	Dry Well Monolithic Porous Pavement Water Quality Inlets

Stormwater Systems of the Future

Stormwater management in urban centers of the U.S. is in the process of metamorphosis. The shift is away from rapid disposal of surface runoff. Instead governing bodies are looking at urban stormwater runoff impacts on the receiving waters and how to minimize these impacts to a “maximum extent practicable.” Urbanization affects the environment, including the nature and quality of the receiving waters. This inescapable fact is driven by population growth. Although some believe that such impacts can be eliminated, the laws of conservation of space, matter and energy consign challenge such beliefs. Therefore, society has to find ways to make wise and cost effective choices to minimize the impact of population growth and its resultant urbanization on the receiving waters. Too ambitious a program can have profound economic impacts on the public and can become economically and politically self defeating. At the same time, doing nothing can have a profound detrimental effect on the receiving waters that also translates to harsh economic impacts on the local public as well.

As much as some may wish it was not so, barring major natural disasters continued urban growth has to be assumed as a given. How stormwater runoff from this growth is managed will define how urban centers will evolve in the next century. The challenge is to find systems and their components that both serve the environment and the needs of the urban communities to the maximum practicable level desired by the U.S. Congress, the individual states and the local municipal populations. Doing this requires learning how to moderate impacts of urbanization on each receiving system as it relates to the local geography, geology and climate, realizing that all impacts cannot be eliminated. At the same time, the systems should not have draconian impacts on urbanization, a natural effect of population growth. With these thoughts as background, the following ideas are offered as possible stormwater management systems of the future.

Use of Combined Wastewater and Storm Sewer Systems

Some have suggested the return to the use of combined wastewater and stormwater systems, that is CSS. The suggestions range from complete coverage of all new urban areas by such systems to the limiting of their use to only high density commercial and industrial areas. Most of these suggestions include detention elements to modulate flow rates into such systems and to limit the size of the conveyance sewers and treatment works. Such systems would result in the first flush of larger storms and all runoff from smaller storms being captured and treated through publicly owned wastewater treatment plants before release to the receiving systems. Much of the stormwater entering headwater streams would be diverted to such systems, thus reducing the impacts of increased stormwater runoff into these streams.

On the other hand, these systems would have occasional combined sewer overflows. In the process of diverting stormwater runoff from the headwater streams, other hydrologic changes will likely occur, such as groundwater depletion and reduced base flows in perennial streams. The biggest drawback to these systems is the cost of their construction, operation, and maintenance. Much bigger sewers would be needed to transport stormwater to a treatment plant, even with detention, than are needed to deliver stormwater to the nearest receiving waterway. The treatment plant also needs much greater capacity to handle the 10 to 30 percent of the days during any given year when wet weather flows actually occur. Combined systems need a much higher level of maintenance than separate sewer and storm sewer systems. Also, these systems will require an increased use of non renewable resources (i.e., electric power, petroleum based fuels and chemicals) to treat stormwater. Whether these added costs are justified will depend on site specific conditions such as the receiving waters and the impacts on them that are being mitigated, the community’s size and economic strength.

With the foregoing in mind one scenario for a stormwater system of the future would consist of a hybrid system, one that serves part of the urban area with a combined wastewater and separate stormwater system and the remaining part with a separate stormwater system. More specifically it would consist of the following:

1. The use of good housekeeping, and non-structural BMPs, is well established and practiced, with especially strong emphasis on control of illegal and illicit discharges of contaminants and the control of erosion during construction.

2. Major facility needs of the stormwater management system would be based on a watershed, or sub-watershed level master planning process. The community would be involved in the process.

3. The process would account for future growth, drainage system and other infrastructure needs of the community and integrate all of these with community needs such as open space, recreation, jobs, and transportation. Impacts, growth trends, costs, maintenance needs, benefits and other issues and needs would be identified and, when possible, quantified.

4. Use of the minimized DCIA elements wherever practicable and possible in residential areas and commercial parts of the community and in areas such as parks, golf courses, playgrounds, playing fields, churches, and recreation centers.

5. An extensive use of surface infiltration and flow retardance elements such as grass buffers, swales, porous pavement, and infiltration basins when site geology and site conditions permit.

6. Extensive use of on site or regional extended detention basins, retention ponds and/or wetland basins for all urbanizing areas, whether connected or not, to the CSS.

7. Sized to capture a water quality volume to also help mitigate increases in surface runoff from small events.

8. When the drainage system and public safety requires, provide for a surcharge flood control detention above the water quality capture volume.

9. All high density commercial areas, gasoline stations, other commercial areas subject to surface contamination by chemicals or high concentrations of nutrients, and industrial areas subject to chemical surface contamination be connected to a combined sewer system.

10. All connections to the CSS would be made through water quality capture volume basins.

11. All releases from the water quality capture basins connected to the CSS would be controlled by an intelligent real-time flow management system designed to meet the conveyance and the treatment plant system's capacities.

Use of Separate Stormwater Systems

Use of a hybrid combined wastewater and stormwater system may not be the best or practical option for the majority of communities in U.S. As discussed earlier, these systems are likely to be more expensive, in terms of life cycle costs, to build and operate than two separate systems, one for wastewater and the other for stormwater.

When a hybrid combined system is not a cost effective or practical solution, what is left is a separate stormwater management system that uses various management and land use development practices to control stormwater runoff quality and quantity as close to the source as practicable. The goal of an ideal separate stormwater management system of the future would be to select stormwater management components that best mitigate the impacts of urbanization on the receiving waters for the community in a most practical and cost effective manner. Similar to the hybrid combined system, a separate stormwater system of the future would capture the first flush of larger storms and all runoff volume from smaller storms. The captured volume would receive passive treatment by the BMP before stormwater is released to the receiving systems within or downstream of the community. Such a system could significantly reduce the impacts of increased stormwater runoff and its contaminants on these receiving waters.

With the foregoing, a possible scenario for a stormwater system of the future is as follows:

1. The use of good housekeeping, non-structural BMPs, is well established and practiced, with especially strong emphasis on illegal and illicit discharges of contaminants and the control of erosion during construction.

2. Major facility needs of the stormwater management system would be based on a watershed, or sub-watershed level master planning process. The community would be involved in the process. The process would account for future growth, drainage system needs and other compatible use needs of the community. Impacts, growth trends, costs, maintenance needs, benefits, and other issues and needs would be identified and, when possible, quantified.

3. Use of minimized DCIA elements wherever practicable and possible in residential areas and areas such as parks, golf courses, playgrounds, playing fields, and recreation centers.

4. An extensive use of surface infiltration and flow retardance elements such as grass buffers, swales, porous pavement, and infiltration basins when site geology and site conditions permit.

5. Extensive use of on site or regional extended detention basins, retention ponds and/or wetland basins for all urbanizing areas.

    - Sized to capture a water quality volume and to also help mitigate increases in surface runoff from small events.

    - When the drainage system and public safety requires, provides for a surcharge flood control detention above the water quality capture volume.

6. All high density commercial areas, gasoline stations, other commercial areas subject to surface contamination by chemicals or high concentrations of nutrients, and industrial areas subject to chemical surface contamination be addressed on a site-by-site basis to reduce stormwater runoff flow rates and contaminants to maximum extent practicable. Some of these sites may need special treatment measures for the pollutants being generated on the site such as special media filters, and chemical additives.

   - All runoff from the areas subject to contamination be routed through water quality capture volume basins. These basins may need to be oversized if the pollutants are of major concern for environmental and public health protection.

   - All such water quality capture basins would be occasionally audited for compliance to insure that the needed operation and maintenance is being provided. Also, occasional grab samples of the effluent would be taken and tested by their owners.

Closing Remarks

This chapter discusses many issues that relate to BMPs and what is known about their effectiveness in stormwater management. Much of this discussion is based on a plethora of information that is "supported" by a number of local field investigations designed to test a given BMP's "effectiveness" at the specific site. Still needed is a national approach, similar to NURP, that would systematize a large number of investigation into a cohesive, well controlled, program to learn about various BMP functions, physical mechanisms, biochemistry, and design parameters.

Also needed is a better measure of "effectiveness. The current measure in terms of "percent pollutant removal" has no sound technical basis. This is the case whether the effectiveness is measured in term of constituent load reductions or in terms of reduction in concentrations. Lack of a sound definition can lead to findings that may appear to be inconsistent and non-transferable, when in truth, the differences may not be that large if a better measure of effectiveness is used. Another area of need is improving on the design robustness for various BMPs. Until that is done, expecting a specific performance from any given BMPs is unrealistic. Design robustness should improve as more is learned about what design parameters are most important when selecting, sizing and designing each type of BMP.

Urban stormwater management has to consider the safety and welfare of the citizens living in urban areas. Issues of efficient site drainage, control of nuisances caused by inadequate drainage, hazards posed by large storm events and the floods they create, and cost and benefits received for the expenditure of public dollars have to be considered along with stormwater quality and impact on the receiving water quality, integrity and biology. As a result, sound stormwater management has to address not only runoff impact mitigation associated with urbanization, but also the public and community needs as well

The preceding discussion summarizes the potential usability of BMPs. All of this is based on information in need of enrichment. Nevertheless, it should provide a basis for understanding the current BMP state of-of-practice and state-of-the-art and, accordingly, serve as a guide for planners and engineers.

References

Anderson, D.L., R.L. Siegrist, and R.J. Otis (undated). Technology Assessment of Intermittent Sand Filters. Office of Research and Development. Environmental Protection Agency. Cincinnati, OH.

Bell, W., L., Stokes, L.J. Gavan, and T. Nguyen (1996). Assessment of the Pollutant Removal Efficiencies of Delaware Sand Filter BMPs. City of Alexandria, Department of Transportation and Environmental Services. Alexandria, VA.

Cerco, C.F. (1995). Response of Chesapeake Bay to nutrient load reductions. Journal of Environmental Engineering. Vol. 121, No. 8.

Chang, G.C., J.H., Parrish, C. Soeur, and A.S. Librach (1990). Removal Efficiencies of Stormwater Control Structures. Dept. of Environmental and Conservation Services. City of Austin, TX.

City of Austin (1988). Environmental Criteria Manual. Environmental and Conservation Services. Austin, TX.

Colorado Storm Water Task Force (1990). BMP Practices Assessment for the Development of Colorado's Stormwater Management Program. Final Report to Colorado Water Quality Control Division. Denver CO.

Day, G.E., D.R. Smith, and J. Bowers (1981). Runoff and Pollution Characteristics of Concrete Grid Pavements. Bulletin 135, Virginia Water Resources Research Center. Blacksburg, VA.

DiToro, D.M. and J.J. Fitzpatric (1993). Chesapeake Bay Sediment Flux Model. Prepared for U.S. Environmental Protection Agency and U.S. Corps of Engineers Baltimore District.

Driscoll, E.D., G.E., Palhegyi, E.W. Strecker, and P.E. Shelley (1989). Analysis of Storm Events Characteristics for Selected Rainfall Gauges Throughout the United States. U.S. Environmental Protection Agency. Washington, DC.

Farnham, R.S. and T. Noonan (1988). An Evaluation of Secondary Treatment of Stormwater Inflows to Como Lake, MN Using A Peat-Sand Filter. EPA Proj. S-005660-02. Environmental Protection Agency. Chicago, IL.

Galli, J. (1990). Peat-Sand Filters: A Proposed Stormwater Management Practice for Urbanized Areas. Metropolitan Washington Council of Governments. Washington, DC.

Glidden, M.W. (1981). The Effects of Stormwater Detention Policies on Peak Flows in Major Drainageways. Master of Science Thesis. Dept. of Civil Engineering. University of Colorado at Denver. Denver, CO.

Grizzard, T.J., C.W., Randall, B.L. Weand, and K.L. Ellis (1986). Effectiveness of extended detention ponds. Urban Runoff Quality - Impact and Quality Enhancement Technology. Urbonas, B.R. and Roesner, L.A., Editors. American Society of Civil Engineers.

Guo, J.C.Y. and B. Urbonas (1996). Maximized detention volume determined by runoff capture ratio. J. Water Resources Planning and Management. Vol. 122, No. 1, p. 33-39.

Hall, M.H., D.L. Hockin, and J.B. Ellis (1993). The Design of Flood Storage Reservoirs. Butterworth Heineman, London.

Hardt, R.A. and S.J. Burges (1976). Some Consequences of Area Wide Runoff Control Strategies in Urban Watersheds. Technical Release No. 48. Charles W. Harris Hydraulics Laboratory. University of Washington. Seattle, WA.

Harper, H. and J. Herr (1992). Treatment Efficiencies of Detention with Filtration Systems. Environmental Research and Design, Inc. Orlando, FL. 164 pp.

Hartigan, J.P. (1989). Basis for design of wet detention basin BMPs. Design of Urban Runoff Quality Controls. American Society of Civil Engineers. New York, NY, p. 122-144.

Heaney, J.P. and L.T. Wright (1997). On integrating continuous simulation and statistical methods for evaluating urban stormwater systems. Modeling the Management of Stormwater Impacts. W. James, editor. Computational Hydraulics International. Guelph, Ontario, Canada. p.44-76.

Kedlec, R.H. and D.E. Hammer (1980). Wetland Utilization for Management of Community Wastewater. 1979 Operations Summary, Houghton Lake Water Treatment Project. NTIS PB80-170061.

Lakatos, D.F. and L.J. Mcnemer (1987). Wetlands and Stormwater Pollution Management. Wetland Hydrology. Proceedings of the National Wetland Symposium. Chicago, IL.

Lindsey, G., L. Roberts, and W. Page (1991). Stormwater Management Infiltration Practices in Maryland: A Second Survey. Maryland Dept. of the Environment. Baltimore, MD.

Livingston, E. H., et al. (1988). The Florida Development Manual: A Guide to Sound Land and Water Management. Department of Environmental Regulation. Tallahassee, FL.

McCuen, R.H. (1974). A regional approach to urban stormwater detention. Geophysical Research Letters. 74-128, p. 321-322. American Geophysical Union. Washington, DC.

Neufeld, C.Y. (1996). An investigation of Different Media for Filtration of Stormwater. Masters Thesis. Department of Civil Engineering. University of Colorado at Denver, CO.

Nichols, D.S. (1983). Capacity of wetlands to remove nutrients from wastewater. Journal of Water Pollution Control Federation. Vol. 55, No. 5. Pp. 495-505.

Oberts, G.L., P.J. Wotzka, and J.A. Hartsoe (1989). The Water Quality Performance of Select Urban Runoff Treatment Systems. Metropolitan Council of the Twin Cities Area. St. Paul, MN. June.

Pensyl, K. and P.F. Clement (1987). Results of the State of Maryland Infiltration Practices Survey. Sediment and Stormwater Div. Maryland Dept. of the Environment. Baltimore, MD.

Pitt, R., M., Liburn, S. Nix, S.R. Durrans, and S. Burian (1997). Guidance Manual for Integrated Wet Weather Flow (WWF) Collection and Treatment Systems for Newly Urbanized Areas (New WWF Systems). Report prepared for U.S. Environmental Protection Agency. Risk Reduction Engineering Laboratory. Office of Research and Development. Cincinnati, OH.

Pratt, C.J. (1990). Permeable pavement for stormwater quality enhancement. Urban Stormwater Quality Enhancement. American Society of Civil Engineers. New York, NY. p. 131-155.

Roesner, L.A., H.E. Burgess, and J.A. Aldrich (1991). The hydrology of urban runoff quality management. Proceedings of a Water Resources Planning and Management Conference. American Society of Civil Engineers. New Orleans, LA.

Roesner, L.A., B.R. Urbonas, and M.B. Sonnen, Editors (1989). Design of Urban Runoff Quality Controls. American Society of Civil Engineers.

Schueler, T. (1987). Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. Metropolitan Washington Council of Governments. Washington, D.C.

Schueler, T.R. and J. Galli (1992). The Environmental Impact of Stormwater Ponds. Effects of Urban Runoff on Receiving Systems. American Society of Civil Engineers.

Schueler, T.R., P.A. Cumble, and M.A. Heraty (1991). Current Assessment of Urban Best Management Practices: Techniques for Reducing Non-point Source Pollution in the Coastal Zones. (Review Draft). Metropolitan Washington Council of Governments. Washington, D.C.

Shaver, E. and R. Baldwin (1991). Sand Filter Design for Water Quality Treatment. Delaware Department of Natural Resources and Environmental Control. Dover, DE.

Smith, D.R. (1984). Evaluation of concrete grid pavements in United States. Proceedings Second Conference on Concrete Block Paving. Delft, Australia. Society of Civil Engineering.

South Florida Water Management District. Technical Publication 88-9. West Palm Besch, FL.

Southwest Florida Water Management District (1995). Proceedings of the 4th Biennial Stormwater Research Conference. Southwest Florida Water Management District (editor and publisher). Brooksville, FL.

Stahre, P. and B. Urbonas (1990). Stormwater Detention for Drainage, Water Quality and CSO Management. Prentice Hall. Englewood Cliffs, NJ.

Stewart, W. (1989). Evaluation and full-scale testing of a compost biofilter for stormwater runoff treatment. Presented at the Annual Conference of the Pacific Northwest Pollution Control Association. Seattle, WA.

Strecker, E.W., G.E. Palhegyi, and E.D. Driscoll (1990). The use of wetlands for control of urban runoff pollution in USA. Proceedings of the Fifth International Conference on Urban Storm Drainage. Osaka, Japan.

Truong, H.V., C.R., Burrell, M.S. Phua, and R.D. Dallas (1993). Application of Washington DC Sand Filter for Urban Runoff Control. District of Columbia Environmental Regulation Administration. Washington, DC.

Urban Drainage & Flood Control District (1992). Urban Storm Drainage Criteria Manual: Volume 3 - Best Management Practices. Denver, CO.

Urbonas, B. and P. Stahre (1993). Stormwater - Best Management Practices and Detention. Prentice Hall. Englewood Cliffs, NJ.

Urbonas, B., L.A., Roesner, and J.C.Y. Guo (1996a). Hydrology for Optimal Sizing of Urban Runoff Treatment Control Systems. Water Quality International, International Association for Water Quality. London, England.

Urbonas, B.R. (1979). Reliability of design storms in modeling. Proceedings, International Symposium on Urban Storm Runoff Modeling. University of Kentucky. Lexington, KY.

Urbonas, B.R. (1983). Potential effectiveness of detention policies. Southwest Urban Stormwater Symposium. Texas A & M. Austin, TX.

Urbonas, B.R. (1995). Recommended parameters to report with BMP monitoring data. J. Water Resources Planning and Management. Vol. 121, No. 1, pp. 23-34.

Urbonas, B.R. (1997). Hydraulic design of sand filters for stormwater quality. Flood Hazard News. Denver, CO.

Urbonas, B.R., and W. Ruzzo (1986). Standardization of detention pond design for phosphorus removal. Urban Runoff Pollution*.* NATO ASI Series Vol. G10. Springer-Verlag, Berlin.

Urbonas, B.R., J. Doerfer, and L.S. Tucker (1996b). Stormwater sand filters: a solution or a problem? APWA Reporter. May Issue. Washington DC.

Urbonas, B.R., C.Y. Guo, and L.S. Tucker (1990). Optimization of stormwater quality capture volume. Urban Stormwater Quality Enhancement. American Society of Civil Engineers.

Urbonas, B.R., B. Vang, and J. Carlson (1993). Joint pond-wetland system performance in Colorado. Water Policy and Management: Solving the Problems. Proceedings of the 21st Annual Conference of the Water Resources Planning and Management Division. The American Society of Civil Engineers. New York, NY.

US EPA (1983). Results of the Nationwide Urban Runoff Program. Final Report. U.S. Environmental Protection Agency. NTIS Access No. PB84-18552. Washington DC.

USGS (1986). Constituent Load Changes in Urban Stormwater Runoff Routed Through a Detention Pond - Wetland System in Central Florida. Water Resources Investigation 85-4310. U.S. Geological Survey. Tallahassee, Fl.

Veenhuis, J., J. Parish, and M. Jennings (1989). Monitoring and design of stormwater control basins. Design of Urban Runoff Quality Controls. American Society of Civil Engineers. New York, NY.

Walesh, S.G. (1986). Case studies of need-based quality-quantity control projects. Proceedings of Urban Runoff Quality-Impact and Quality Enhancement Technology. An Engineering Foundation Conference. American Society of Civil Engineers. New York, NY.

Walesh, S.G. (1997). DAD (decide-announce-defend) is out, POP (public owns project) is in. Water Resources Education Training and Practice: Opportunities for the Next Century. American Water Resources Association. Keystone, CO. June 1997.

Walesh, S.G. (1993). Interaction with the public and government officials in urban water planning. Hydropolis – The Role of Water in Urban Planning. Proceedings of the International UNESCO-IHP Workshop. Wageningen, The Netherlands and Emscher Region, Germany. March – April.

Walesh, S.G. (1989). The hydrologic cycle in the urban environment. Chapter 2 in Urban Surface Water Management. John Wiley and Sons. New York, NY.

Wanielista, M.P., Y.A., Yousef, H.H. Harper, and C.L. Cassagnol (1981). Detention with effluent for stormwater management. Proceedings of the 2nd International Conference of Urban Stormwater Management. Urbana, IL.

Watson, T.J., S.C., Reed, R.C., Kadlec, R.L. Knight and A.E. Whitehouse (1989). Performance Expectation and Loading Rates for Constructed Wetlands. Constructed Wetlands for Wastewater Treatment: Municipal, Industrial, and Agricultural. Lewis, Chelsea, MI.

WEF & ASCE (1998). Urban Runoff Quality Management. Water Environment Federation Manual of Practice No. 23 and American Society of Civil Engineers Manual and Report on Engineering Practice No. 87. WEF. Alexandria, VA. ASCE. Reston, VA.

Whallen, P.J. and M.G. Cullum (1988). An Assessment of Urban Land Use: Stormwater Runoff Quality Relationships and Treatment Efficiencies of Selected Stormwater Management Systems.

Whipple, W. and J.V. Hunter (1981). Settleability of urban runoff pollution. Journal, Water Pollution Control Federation. Vol. 53, No. 12, pp. 1726-1731.

Wiegand, C., T. Schueler, W., Chittenden, and D. Jellick (1989). Cost of urban quality controls. Design of Urban Runoff Quality Controls. American Society of Civil Engineers. New York, NY.

Chapter 8

Stormwater Storage-Treatment-Reuse Systems

James P. Heaney, Len Wright, and David Sample

Introduction
The overall effectiveness of a variety of stormwater BMP's was evaluated in Chapter 7. Two other aspects of control of stormwater: high-rate treatment and the potential effectiveness of using stormwater for supplemental irrigation are described in this chapter.

Stormwater Treatment
Because of the dynamic nature of stormwater flows and water quality, most control systems are a hybrid of temporary storage and high-rate treatment. For a given level of stormwater control, the engineer can accomplish this objective using various combinations of storage and treatment. Much has been written on this subject and methods for finding the optimal combination of storage and treatment have been developed. Heaney and Wright (1997) provide a summary of these methods. Several unresolved issues remain with regard to evaluating the performance of these treatment systems.

Effect of Initial Concentration
As pointed out in Chapter 7, the effect of initial concentration on the performance of wet-weather controls should not be ignored. A high percent removal for a control will usually occur if the initial concentration is high. Separate and combined stormwater flows exhibit wide variability from storm to storm as well as within a given storm. The effect of initial concentration on performance can be evaluated directly by finding the order of the reaction as well as the rate constant (Heaney and Wright 1997).

Effect of Change of Storage
Another complication in dealing with treatment of wet-weather flows is that the control units are typically filling and emptying during and following the storm. Thus, it is vital to properly measure the change in storage at short time intervals to incorporate this important factor. The effect of changing storage is captured in the calculated detention time for each parcel of water.

Effect of Mixing Regime
Another critical assumption is the type of mixing that takes place in the treatment reactor. Two limiting cases are plug flow wherein the parcels simply queue through the reactor and complete mixing wherein the incoming parcel instantaneously mixes with the water already in the reactor.

Effect of Nature of the Suspended Solids

The nature of the suspended solids changes during the storm and can vary widely. The solids can range over several orders of magnitude from coarse solids to fine colloids. Pisano and Brombach (1996) present a summary of efforts to date to characterize wet-weather solids.

Essential Features of Future Wet-Weather Control Facilities

Given the large variability in the quantity and quality of wet-weather flows and the filling and emptying of treatment reactors, direct monitoring of the wet-weather inflows and the status of the control units is of fundamental importance. Unfortunately, few such systems have been built in the United States. The Europeans are more advanced in trying to evaluate and optimize wet-weather control systems.

High-Rate Operation of Wastewater Treatment Plants

High-rate operation of WWTPs during and following wet-weather events is an important option to evaluate as part of the overall stormwater management program for combined and for separate systems that are affected by I/I. It is possible to model the expected performance of these systems using the GPS-X WWTP software from Hydromantis, Inc., or similar programs, to do continuous simulation of the effect of wet-weather flows on DWF treatment plants. Mangeot (1996) performed a preliminary feasibility study using GPS-X to evaluate the Boulder WWTP during the 1995 high-flow year. High-rate operation of the WWTP during these wet periods and periods with high I/I due to seasonably high groundwater tables appears to be a very attractive option to consider. Not much research has been done on this problem and there are only a few literature citations on results of attempting to model the dynamics of WWTP operation during high flow periods. Some questions remain regarding the ability of GPS-X to properly handle the hydraulics associated with wet-weather flows. However, it is possible to show with direct measurements for the Boulder WWTP, that the plant is capable of operating effectively over a wide range of influent flows and concentrations. Because the influent is already so dilute, caution should be exercised in requiring a specified percent removal under these wet-weather conditions.

Stormwater Reuse Systems

Introduction

At present, there is much interest in local management of stormwater from smaller, more frequent events. The primary on-site option is to encourage infiltration of stormwater from roofs, driveways, parking lots, and streets. This infiltrated water increases the moisture in the unsaturated zone and raises the groundwater table which can provide benefits in terms of increasing base flows in streams and providing storm water to help meet the ET needs of the local vegetation. Higher groundwater levels can have negative effects on basements and on sanitary and combined sewers. This section explores the possibility of the reuse of urban stormwater for irrigation water which is a major component of urban water use.

Previous Studies

As water supplies become more stressed, water conservation and reuse become more attractive options. Wastewater disposal costs also encourage more water reuse. Asano and Levine (1996) provide a historical perspective and explore current issues in wastewater reclamation, recycling, and reuse, and outline requirements of a stormwater and wastewater reuse feasibility study. Lejano et al. (1992) summarize the benefits of water reuse as the following:

1. Water supply related:
   a. Supplements regional water supply, eliminating need to develop additional supplies.
   b. Provides more reliability than the usual supply and is less affected by weather.
   c. Provides a locally controlled supply, reducing dependence on state or regional politics.
   d. Avoids the operating costs of water treatment and delivery.
   e. Eliminates social and environmental impacts of diverting water from natural drainageways.
   f. Eliminates impacts of constructing large-scale water storage and transmission facilities.

2. Wastewater related:
   a. Avoids the capital and operating costs of disposal facilities.
   b. Avoids the costs of advanced treatment facilities needed to meet state and federal discharge requirements.

Urban wet weather flow management needs to be viewed within the context of overall urban water management. Such an integrated framework was proposed in the late 1960s and is regaining favor in the mid-1990s. Changes in urban water use are occurring because of aggressive water conservation practices which will significantly reduce indoor and outdoor water use.

As discussed in Chapter 3, per capita indoor residential water use is very stable at an average of 60 gpcd. Aggressive hardware changes such as low flush toilets should reduce this usage rate to 35-40 gpcd. Only a small proportion of this indoor waste is black water. Most of it is graywater that could be reused on-site for lawn watering and other non-potable purposes. Peak water use in most cities is heavily influenced by urban lawn watering. This outdoor water use does not require potable quality. As the cost of water treatment continues to increase, dual water systems become more of a possibility, particularly with a decentralized infrastructure.

California has been a focal point of reuse activity for some time. Ashcraft and Hoover (1991) found that reclaimed water in southern California is selling at prices ranging from \$303/ac-ft to \$366/ac-ft, with costs of operation and maintenance of treatment facilities running from \$10/ac-ft to \$95/ac-ft. The authors argue that “avoided costs,” such as those associated with wastewater disposal should be included in cost calculations.

Mallory and Boland (1970) developed a hydrologic and economic optimization model of a stormwater reuse system in a new town in Maryland. Their system used a network of subdivision level detention ponds. Subpotable reuse required a dual distribution system to deliver it to households. They found that the net capital cost of such a system (scaled up to 1998 dollars) was $560/dwelling unit for a potable reuse system, and $1175/dwelling unit for the subpotable system. This compares favorably with $950/dwelling unit for a conventional system, the differential of 23% premium for subpotable reuse due mainly to the dual distribution system. When pollution control costs are included for stormwater quality, an additional cost of $640/dwelling unit was calculated, making the investment in the subpotable system more attractive.

Requa et al. (1991) developed a wastewater reuse cost model for screening purposes in northern California. More recently, Tselentis and Alexopoulou (1996) describe a feasibility study of effluent reuse in the Athens, Greece metropolitan area. Uses considered were: crop irrigation, irrigation of forested areas, industrial water supply and domestic non-potable use. The most cost-effective scenario was distribution for crop irrigation near the route of the current discharge point.

At the other extreme, Haarhoff and Van der Merwe (1996) describe direct potable reuse of reclaimed wastewater in Windhoek, Namibia. Law (1996) describes the Rouse Hill project in Sydney, Australia, in which a dual non-potable distribution system was installed in a new community in 1994. Oron (1996) developed an integrative economic model, arguing that the optimal cost of a reuse system is a function of treatment method, cost of treatment, transportation and storage costs (pipelines and tanks), environmental costs, and the selling price of reused wastewater. New initiatives for reusing stormwater flows for urban residential and industrial water supply systems in Australia were described by Anderson (1996a, 1996b).

Mitchell, Mein, and McMahon (1996) used a water budget approach to integrate storage and reuse of urban stormwater and treated wastewaters for two neighborhoods in suburban Melbourne, Australia. The authors developed an urban water balance model to determine the impact of stormwater and wastewater reuse; and suggest its application at a number of scales. They determined that water demand from reservoirs in Australia could be halved through the use of this resource.

Nelen, DeRidder, and Hartman (1996) described the planning of a new development for about 10,000 people in Ede, Netherlands that considers a dual water supply system. Storing the treated wastewater on-site during wet weather periods can be more attractive than only using black water for reuse (Pruel, 1996). Herrmann and Hase (1996) described rainwater utilization systems in Bavaria, Germany that save drinking water and reduce roof run-off to the sewerage system. The impact of urbanization on the hydrological cycle of a new development near Tokyo, Japan was performed by Imbe, Ohta, and Takano (1996).

Much of this work has focused upon using treated wastewater from a single effluent plant. The problem then becomes one of finding demand centers for the wastewater that are typically located quite some distance away. This becomes a nonlinear form of the transhipment problem, in which demand and distance are cost drivers in a nonlinear objective function.

Many researchers have started to focus on less centralized systems, including Tchobanoglous and Angelakis (1996). Decentralized systems can take advantage of the segregation between wet weather flow, graywater, and blackwater, and possibly utilize less contaminated waters closer to their points or origin. Of the three, stormwater runoff is usually the least contaminated prior to central collection. This may avoid construction of additional treatment systems, pipelines, and other infrastructure and present significant cost savings.

From the wet weather flow quality management perspective, there is much interest in local management of wet weather flow from smaller, more frequent events, as these events tend to have more pollutants associated with them. The primary on-site option is to encourage infiltration of this stormwater flow from roofs, driveways, parking lots, and streets.

Herrmann et al. (1996) found that rainwater utilization (using roof runoff water directed into a storage tank) could provide from 30-50% of total water consumption of a residence and reduce heavy metals (in stormwater runoff not reused) by 5-25%. Wanielista (1993) developed design curves in order to determine the storage retention volumes necessary to achieve given proportions of reuse. The design curves are based on a daily water-balance model. The main objectives for this practice in the State of Florida are the costs avoided of using municipal or pumped groundwater for irrigation purposes. From the regulatory viewpoint, the main objective is to discharge some of the stormwater onto the land and thereby get credit for 100% removal of this pollutant source.

Field (1993) did a cost-effectiveness study of the reuse of urban stormwater to meet a variety of differing demands for a hypothetical urban area. The proposed uses varied in their water quality needs, as did the corresponding treatment system designated for that use. Nowakowska-Blaszcyzyk and Zakrzewski (1996) project increases in suspended solids, nitrates, COD, BOD, and lead from rainfall routed through the following sources: roofing, parking areas, streets, storm sewers, infiltration through lawns, and infiltration through sand. The lowest values tended to be from roof runoff. Karpiscak, Foster, and Schmidt (1990) detail the application of stormwater and graywater reuse techniques at a single residence in Tucson, AZ.

Harrison (1993) developed a spreadsheet model to estimate the amount of stormwater captured in a detention pond that could be reused for irrigation in Florida. His work is an application of earlier work by Harper (1991). The Southwest Florida Water

Management District is interested in stormwater reuse as a way of increasing the treatment efficiency of detention systems. Their current design calls for storing the first inch of runoff and draining the pond over a five-day period. They are considering going to an average residence time of 14 days to improve performance from removal rates of 50 to 70 % with a five-day drawdown time. Reusing stormwater would give them a 100% treatment efficiency.

Harrison (1993) uses a daily water budget to estimate the amount of captured urban runoff that could be used for irrigation. The basic storage equation is:

$$\frac{dS}{dt} = R + P + F - RU - D - ET \qquad \text{Equation 8.1}$$

where

$\frac{dS}{dt}$ = the change in storage.

R = runoff volume.
P = direct precipitation onto the pond.
F = water inflow through sides and bottom of the pond which can be negative.
RU =reuse volume.
D = pond outflow.
ET = pond evapotranspiration.

Harrison assumes that there is no net subsurface flow into or out of the pond, i.e., $F = 0$. All values are converted into inches over the equivalent impervious drainage area. A daily time step is used. A minimum precipitation volume of .04 inches is assumed to be needed to produce runoff. This method is identical to the STORM-type calculations with the exception that STORM uses an hourly time step and, in this case, outflows occur either by reuse or direct discharge of the excess water. Harrison does not indicate what he assumed for a pond drawdown rate in addition to the irrigation release. The final results are expressed as a production function showing the percent of the irrigation demand that is satisfied for various combinations of pond size and irrigation reuse rates. The primary purpose of the stormwater reuse study in Florida was to minimize the pond outflow and thereby achieve increased pollutant removal efficiency by infiltrating the water locally. Lawn watering was more of a by-product.

Courtney (1997) explored the potential effectiveness of stormwater runon systems for meeting irrigation needs in Boulder, CO. She used an hourly simulation model that mimicked the operating policy of the University of Colorado's automatic irrigation system. The overall imperviousness of the campus is about 60% so there is ample opportunity for infiltrating some of this storm water. The results of this study indicate that, while much of the stormwater can be infiltrated, it is unclear how much of this water will ultimately be used to satisfy ET. During and immediately following the storm, the ET needs have already been satisfied. Without detailed concurrent groundwater and soil moisture monitoring data, it is not possible to estimate the longer term fate of this captured stormwater. If this stormwater could be directed to local or regional storage ponds, it could be reused later for irrigation. Some of this reuse already happens on the University of Colorado at Boulder campus because some of the stormwater drains to the local irrigation ponds.

Estimating the Demand for Urban Irrigation Water

Urban Water Budgets

One of the most prevalent themes advanced in the recent literature in stormwater management is to limit the generation of runoff from urban areas through the use of BMPs and on-site control of stormwater particularly in frequent small storm events (Mitchell et al. 1996). This section evaluates residential on-site control.

Butler and Parkinson (1997) suggest that reuse of the stormwater resource provides for a more sustainable urban drainage infrastructure by minimizing available stormwater that could possibly be mixed with wastewater; as well as attempting to minimize the use of expensive drinking water for irrigation purposes. Pitt et al. (1996) suggests that residential stormwater (i.e. roofs and driveways, not streets) generally has the least amount of contamination and advocates infiltration of residential stormwater as a means of disposal with few environmental impacts.

In keeping with this theme, a possible model of a residential on-site control system is shown in Figure 8-1. Precipitation falls on roofs and driveways and is channeled, with some losses, into a storage tank. The storage tank varies in size depending upon the location. Water is taken from the tank for irrigation of landscape surfaces; some is used for evapotranspiration, some is lost to infiltration, and some is lost to runoff. In essence, this model is an irrigation, or water deficit demand, model.

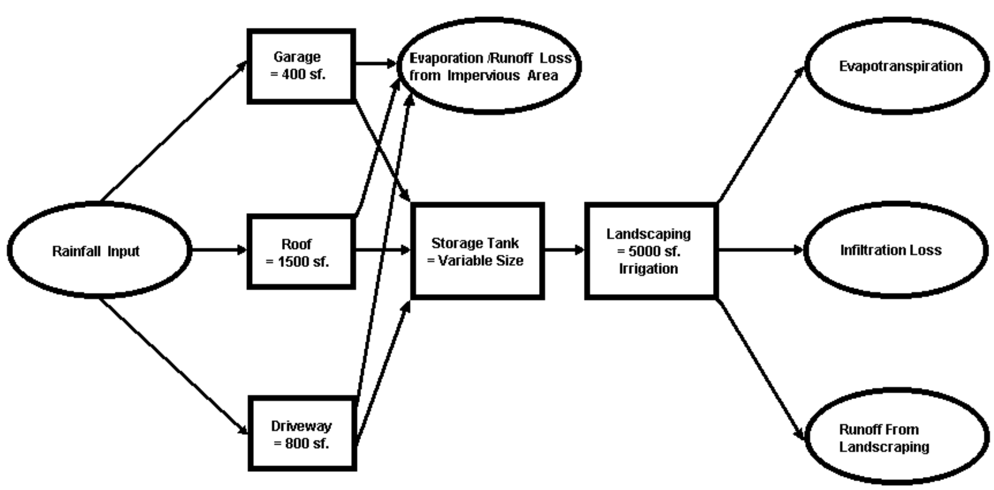


Figure 8-1. Concept of stormwater reuse residential storage system.

Irrigation demand is determined mainly from ET requirements. In order to calculate ET, daily or monthly water budgeting is performed. By examining the water balance of one residential parcel in differing climatic zones, the efficacy of the option of on site reuse of stormwater can be evaluated across the U.S. This section introduces the reader to climatic water balance models, and existing databases for use with these models, develops a parcel level storage/demand analysis using the results from the climatic model and compares results regionally across the U.S.

Water Budget Concepts

The early efforts by Thornthwaite (1948) may have been the first work in climatology in which, by an analytical method, differing characteristics such as rainfall, temperature, and the number of daylight hours in a day were combined to yield regional climatic projections. The number of daylight hours in a day are a function of the latitude of the location, whereas monthly precipitation and temperature are functions of the climate of the location. Average monthly precipitation in the U.S. varies widely with location, as can be seen in Figure 8-2. For example, in comparing the rainfall signature of San Francisco, CA with Memphis, TN, San Francisco has dry summers and wet winters; whereas Memphis appears to have wet springs, with some precipitation falling in every month of the year. Extreme monthly precipitation is also shown in Figure 8-2. San Francisco appears to have much less variability than Memphis.

The Thornthwaite method keeps track of precipitation, calculated potential evapotranspiration (PET), and calculated actual ET on a daily or monthly basis, calculating water deficit, water surplus, soil moisture recharge, and soil moisture utilization by integrating areas under the plotted curves. The graphical representation of this process is a water budget, examples of which are plotted in Figure 8-3, compiled from Mather (1978).

For example, for San Francisco, in January, the precipitation far exceeds the PET (and ET, at this point they are equal). Up until mid February, the soil moisture is being recharged. This occurs until soil moisture capacity is reached, then the rest of the rainfall exceeding PET is surplus (and available for runoff). For San Francisco, the annual surplus is about 4.3 inches. When PET exceeds rainfall (and is greater than ET) in April through October, there are two integrals of importance; the area between PET and ET is the water deficit, or 10.1 inches, and the area between ET and precipitation is what is being drawn from the soil moisture storage. Then, in October, when precipitation exceeds PET, the area between the precipitation curve and PET goes to soil moisture recharge. The annual total PET for San Francisco is 26.6 inches, ET is 16.6 inches, and precipitation is 20.8 inches. Memphis, also shown in Figure 8-3, has an annual total PET of 39.2 inches, ET of 32.5 inches, precipitation of 45.8 inches, a water deficit of 6.7 inches, and a surplus of 13.2 inches. It is readily apparent that the climate, and the subsequent irrigation needs for each location, are significantly different.

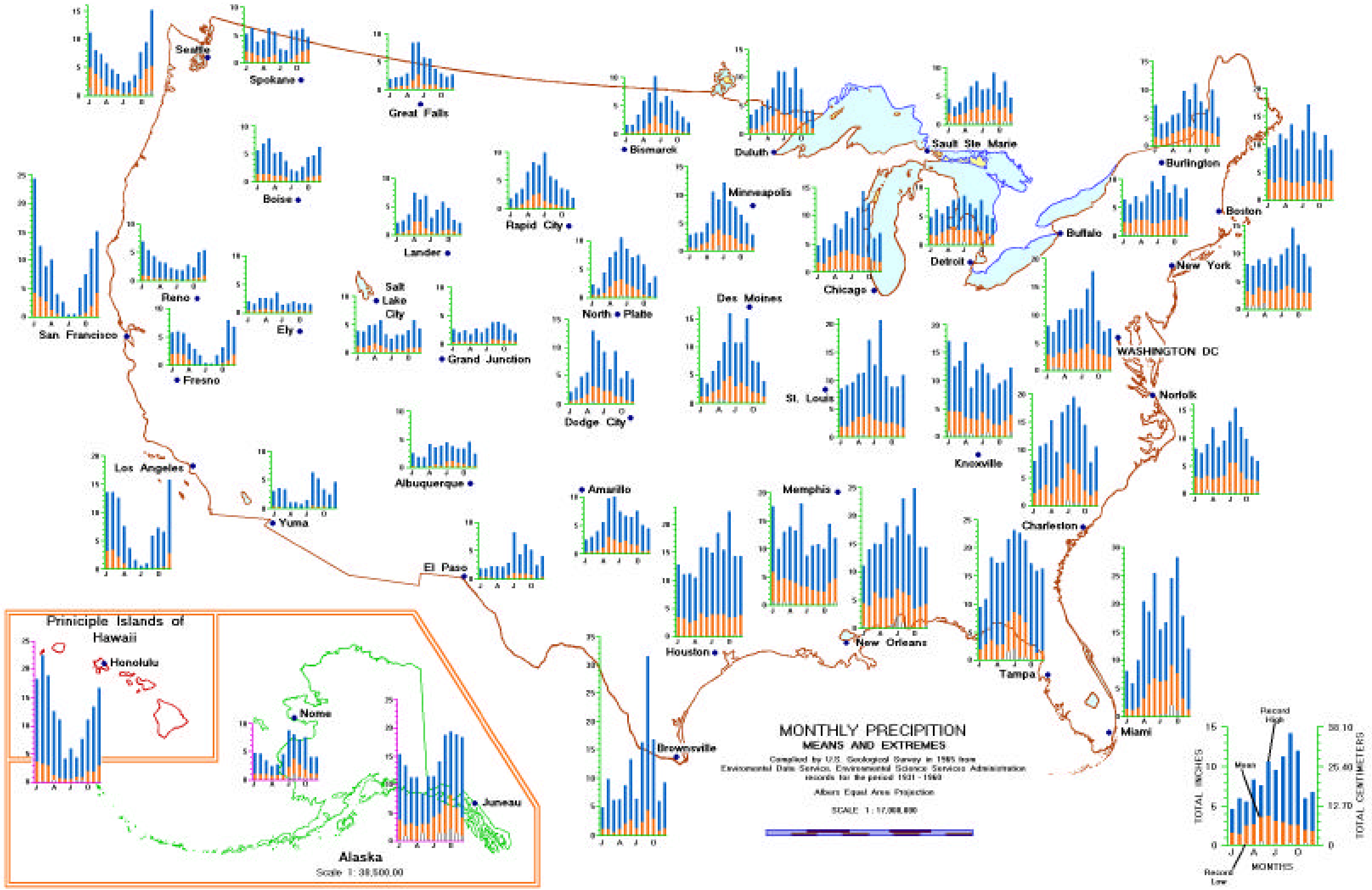


Figure 8-2. Monthly precipitation for selected stations in the U.S., means and extremes (USGS 1970).

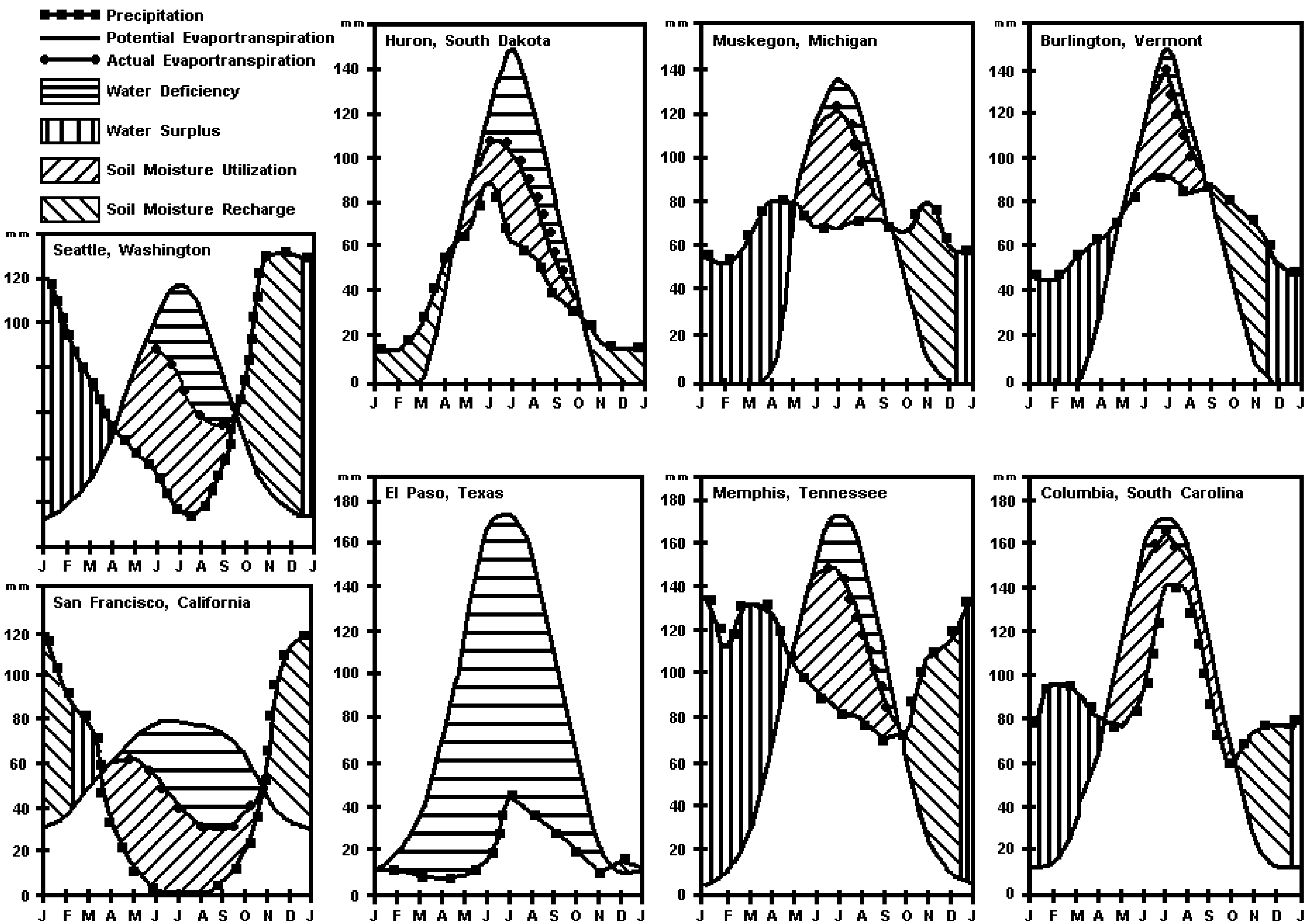


Figure 8-3. Water budgets for selected stations in the U.S. (Mather 1978).

Methods of Analysis

The Thornthwaite and Mather temperature based method (Thornthwaite 1948, Mather 1957, and Willmott 1977) was used to calculate monthly PET, projected ET, water deficit, water surplus, and runoff (for undeveloped areas). Other methods, developed later, require more information, such as net radiation measurements, wind speed, or humidity. Such methods are usually found to be more accurate in arid areas (Yates 1996). An even better approach to the daily water balance model is suggested by Vorosmarty et al. (1996) and explained in detail in Vorosmarty et al. (1989, 1991). This work is a continuation of the work of Mather and Thornthwaite at the University of Delaware.

In the work in this section, the Thornthwaite (or other temperature or radiation based PET model) is used as above, but the soil moisture term is actually modeled as well as the PET. The result is a series of coupled differential equations that are solved by a Runge Kutta algorithm. The input data then reduced to soil and vegetation type. The Thornthwaite method was chosen for this analysis because of the simplicity of the algorithm, as well as the availability of both monthly and daily precipitation and temperature data. Daily data are available for most locations from the National Climatic Data Center.

The water budget procedure is presented in Table 8-1 and graphically in Figure 8-4 for San Francisco, CA. The reader may use the table to follow along the calculations step by step. The mean precipitation, mean temperature, and mean PET (for comparative purposes) are input parameters, and can be found in rows 10, 11, and 29, respectively.

The first step is the calculation of the Julian Day Number. This was done by starting with the number 15 and adding 30 to each successive month in row 12. Next, the geodesic variables are calculated by the following formula:

$$\boldsymbol{f} = 2\boldsymbol{p}\left[Latitude\right]/360 \qquad \text{Equation 8.2}$$

and

$$\boldsymbol{d} = .4093\sin\left[(2\boldsymbol{p}/365)J - 1.405\right] \qquad \text{Equation 8.3}$$

where $\boldsymbol{f}$=latitude in radians in Equation 8.2, $\boldsymbol{d}$ also in radians, is the earth-sun declination angle in Equation 8.3, and J is the Julian day number (e.g., December 31=365). These formulas are used in rows 12 and 13. Next the following term is calculated:

$$\boldsymbol{w}_s = \arccos\left[-\tan\boldsymbol{f}\tan\boldsymbol{d}\right] \qquad \text{Equation 8.4}$$

using the terms calculated above. $\boldsymbol{w}_s$ is the sunset hour angle in radians (Equation 8.4). This is calculated for each month in row 15. Next, the total day length in hours is calculated in Equation 8.5 as follows:

$$N_i = 24\boldsymbol{w}_s/\boldsymbol{p} \qquad \text{Equation 8.5}$$

Table 8-1. Water budget calculations for San Francisco, CA.

	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
8	**Meteorological variable**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Sum (mm)**	**Sum (inches)**
9	Days in month	31	28	31	30	31	30	31	31	30	31	30	31	365	
10	Mean Precipitation, mm	116	93	74	37	16	4	0	1	6	23	51	108	529	20.8
11	Mean Temperature, mm	10.4	11.7	12.6	13.2	14.1	15.1	14.9	15.2	16.7	16.3	14.1	11.4		
12	Julian Day Number	15	45	75	105	135	165	195	225	255	285	315	345		
13	delta, radians	-0.37	-0.24	-0.05	0.16	0.33	0.41	0.38	0.26	0.06	-0.14	-0.31	-0.40		
15	Omegas, radians	1.26	1.38	1.53	1.70	1.84	1.91	1.89	1.77	1.62	1.46	1.32	1.23		
16	Ni, hours	9.64	10.53	11.72	12.96	14.02	14.60	14.41	13.56	12.38	11.14	10.05	9.43		
17	I	3.03	3.62	4.05	4.35	4.80	5.33	5.22	5.38	6.21	5.98	4.80	3.48	57.50	
18	alpha	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39	1.39		
19	**Thornthwaite Model:**														
20	Thornthwaite PET, mm	30	35	48	55	67	75	75	72	73	65	47	34	677	26.6
21	P-PET, mm	86	58	26	-18	-51	-71	-75	-71	-67	-42	4	74	-148	-5.8
22	Storagei, mm	150	150	150	133	94	59	36	22	14	11	15	89	924	36.4
23	Change in storage, mm	60	0	0	-17	-39	-35	-23	-14	-8	-4	4	74	-1	0.0
24	Calculated ET, mm	30	35	48	54	55	39	23	15	14	27	47	34	420	16.6
25	Water Deficit, mm	0	0	0	1	13	35	52	58	59	39	0	0	256	10.1
26	Water Surplus, mm	26	58	26	0	0	0	0	0	0	0	0	0	109	4.3
27	Runoff, mm	26	58	26	0	0	0	0	0	0	0	0	0	109	4.3
28	P-ET, mm	86	58	26	-17	-39	-35	-23	-14	-8	-4	4	74		
29	Measured PET, mm	31	35	49	59	70	78	79	77	75	66	48	35	702	27.6
30	Initial Storage, mm	90													
31	Storage Maximum, mm	150													
32	Error, mm	1	0	1	4	3	3	4	5	2	1	1	1	26	
33	% error													3.68%	

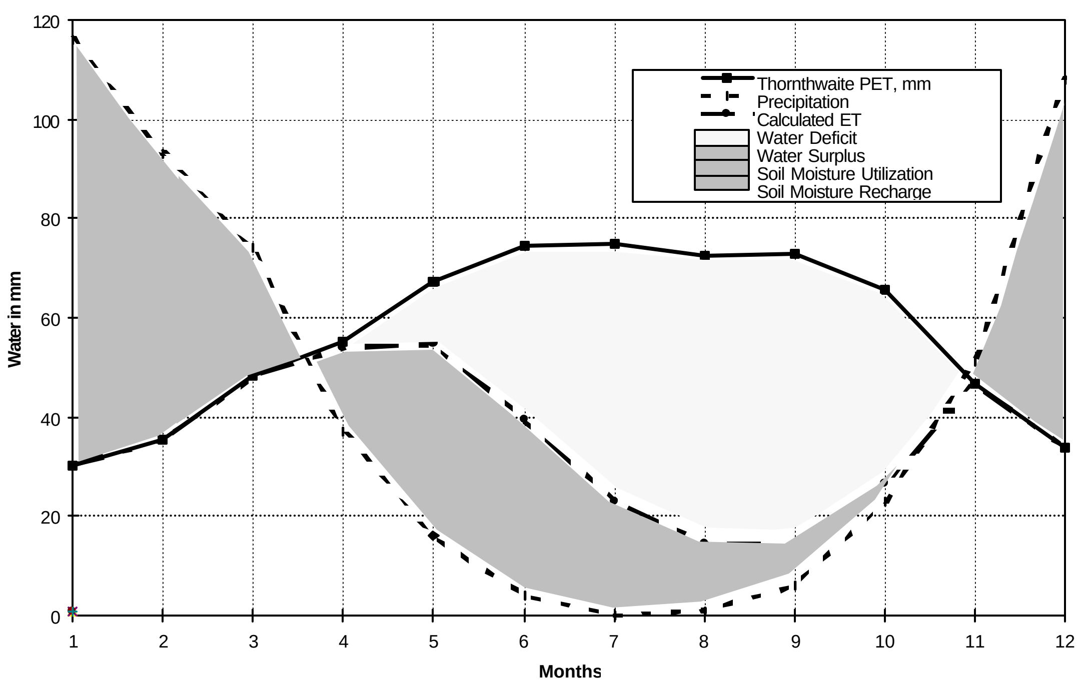


Figure 8-4. Water budget for San Francisco, CA.

and is shown in row 16. Then the following parameters are calculated in Equations 8.6 and 8.7:

$$I = \sum_{i=1}^{n} \left[.2T_i \right]^{1.5} \qquad \text{Equation 8.6}$$

$$a = (6.75*10^{-7})I^3 - (7.71*10^{-5})I^2 + (1.79*10^{-2}) + .49 \qquad \text{Equation 8.7}$$

where n= number of months (or days) in question. These are calculated in rows 17 and 18, the sum of I is calculated by adding all the values of I for the previous 12 months shown in row 17 and is shown in cell P17 . Since T_i (temperature) can be negative, in those cases, I and PET are set to zero. I represents an annual heat index for the area in question. Then, actual values for potential evapotranspiration, PET, storage, S, evapotranspiration, Et, and undeveloped runoff, R are calculated using the Equation 8.8:

$$PET_i = 16 f_1 f_2 \left[\frac{10T_i}{I} \right]^a \qquad \text{Equation 8.8}$$

where $f_1 =$ the fraction of the number of days in the month i divided by the average days in a month, 30; and $f_2 = \frac{N_i}{12}$, the fraction of the number of hours in a day divided by the base of 12 hours in a day. This is calculated in row 20. Next, the soil moisture storage is calculated. This is not to be confused with tank storage, which will be calculated later. The soil moisture storage is modeled as an offline reservoir that leaks when the soil moisture field capacity is reached. Equations 8.9 and 8.10 compute storage in month i as follows:

$$S_i = \min\left[((P_i - PET_i) + S_{i-1}), S_{\max} \right] \text{ if } P_i > PET_i \text{(surplus condition)} \qquad \text{Equation 8.9}$$

$$S_i = S_{i-1} \exp\left[\frac{(PET_i - P_i}{S_{\max}} \right] \text{ if } P_i \le PET_i \text{ (deficit condition)} \qquad \text{Equation 8.10}$$

in which S_i is the soil moisture storage term for month i, P_i is precipitation term for month i, and S_{max} is the maximum storage availability found in cell D31. The initial storage term for month 0 is found in cell D30. The calculated S_i for each month is found in row 22. The change in storage, or $\Delta S = S_i - S_{i-1}$ is calculated in row 23. Next, actual evapotranspiration is calculated by Equations 8.11 and 8.12:

$$Et_i = PET_i \text{ if } P_i > PET_i \qquad \text{Equation 8.11}$$

$$Et_i = P_i + S_{i-1} - S_i \text{ if } P_i \le PET_i \qquad \text{Equation 8.12}$$

and can be found in row 24. Finally, runoff is computed by Equation 8.13,

$$R = P_i - Et_i - \Delta S \qquad \text{Equation 8.13}$$

and is shown in row 27. In cases in which *R<0*, runoff is then set to zero.

The parameters for which the least amount of information is usually available are the initial storage term (when *i*=1) and the maximum soil moisture storage. In this case, an equal S_{max} of 150 mm was used and the initial storage term was determined by using the calculated S_i for December (and iterating if necessary). Water deficit was calculated by subtracting the estimated ET from the calculated PET in months in which PET exceeds rainfall (otherwise there is no deficit). This is shown in row 25. Water surplus was calculated by Equation 8.14:

$$SU_i = P_i - PET_i - \Delta S_i \text{ if } P_i > PET_i \qquad \text{Equation 8.14}$$

and is shown in row 26. The percent error is calculated by taking the absolute value of the difference between the calculated PET and measured PET, summing for the 12 months, and dividing by the sum of the measured PET for 12 months, and is shown in cell P33. For San Francisco, the error is 3.68%, indicating that there is a reasonably good fit with the Thornthwaite model.

The tank calculations for San Francisco are shown in Table 8-2. Using a parcel size of 10,000 sq. ft. (cell D36), and a 1500 sq. ft. house (cell D37), 400 sq. ft. garage(cell D38), an 800 sq. ft. driveway (cell D39), and an irrigated area of 5000 sq. ft. (cell D40), an irrigation demand model was developed in which 80% of the runoff from the house, garage, and driveway was recovered into a storage tank (unless spilled), converting mm of runoff into gallons by multiplying by the impervious areas and conversion factors. This is shown for each month in row 42. These criteria are approximately equal to the dimensions used in the “Casa Del Agua” house in Tucson, AZ (Foster, et al.,1988 and Karpiscak et al., 1990). For purposes of this exercise, runoff from the roof, garage, and driveway are assumed to be channeled into the proposed cistern, which is assumed to be 80% efficient at capturing rainfall (which is consistent with the “Casa Del Agua” case). An initial guess of 100 gallons was given for the storage tank to initiate the calculations.

Water requirements of the landscaped vegetation were assumed to be similar to that predicted by the deficit calculations using the Thornthwaite procedure and losses due to runoff and infiltration were considered negligible. The cumulative volume was then calculated, assuming that the tank initially is empty and that cumulative volume cannot exceed the size of the storage tank, subtracting actual use in the previous month from the storage volume. This is shown in row 43. Next, the potential use or demand for the water was calculated by multiplying the deficit by the irrigated area and converting the number into gallons. This is shown in row 44. The actual use from the storage tank, shown in row 45, is equal to the potential use if it does not exceed the cumulative volume. This procedure is followed in the Table 8-2 for San Francisco.

Table 8-2. Water storage tank calculations for San Francisco, CA.

	C	D	E	F	G	H	I	J	K	L	M	N	O	P
35	**Stormwater calculations:**													
36	size of lot, square footage	10000												
37	square footage of house	1500												
38	square footage of garage	400												
39	square footage of drive and sidewalk	800												
40	square footage of landscaping	5000												
41	Size of tank, gallons	14311												
42	Urban Runoff into tank, gallons	6149	4930	3923	1961	848	212	0	53	318	1219	2703	5725	
43	Cumulative volume, gallons	6149	11079	14311	14311	14184	12741	9995	7978	7222	7089	9528	14311	
44	Potential Use from tank, gallons	0	0	0	127	1570	4316	6333	7089	7222	4783	0	0	31439
45	Actual Use from tank, gallons	0	0	0	127	1570	4316	6333	7089	7222	4783	0	0	31439
46	Difference													0
47	% used													100%

Next, the potential use and actual use are summed for the 12 month period and the difference taken (cell P46). The percentage of the resource used is in cell P47. Because the objective is to maximize the use of the stored stormwater volume, this difference is minimized by successfully selecting larger volumes until the difference is zero or remains constant. In cases in which the difference is zero, the EXCEL function GoalSeek may be used to simplify iterations. If the difference remains constant and not zero, it indicates that it is not possible to meet 100% of the irrigation demand with the available storage, regardless of the tank's volume.

The volume calculated is based upon historically averaged rainfall in a month; a perhaps more accurate method would be to use daily temperature and rainfall data to develop a daily PET model, using several years of data, after developing an autocorrelation model for the precipitation input, and do a Monte Carlo analysis. This would enable the user to capture droughts and probably increase the size of the tank to achieve a greater degree of reliability.

Results

The methodology outlined in the previous section was applied to the cities shown in Figure 8-5. The user can easily create a new worksheet for any city not shown, and copy the database information into it. Then the user may copy the bottom part of any of the existing worksheets containing the model, adjust the initial storage and the latitude to the desired location, and iterate the solution on the tank size, following the procedure in the previous section. By plotting PET, precipitation, and projected ET over the year, and then comparing these numbers to the water deficit, water surplus, and soil moisture storage data, an illustrative plot of the average climatology of a location can be done. Such a plot is given for the city of San Francisco, CA in Figure 8-5.

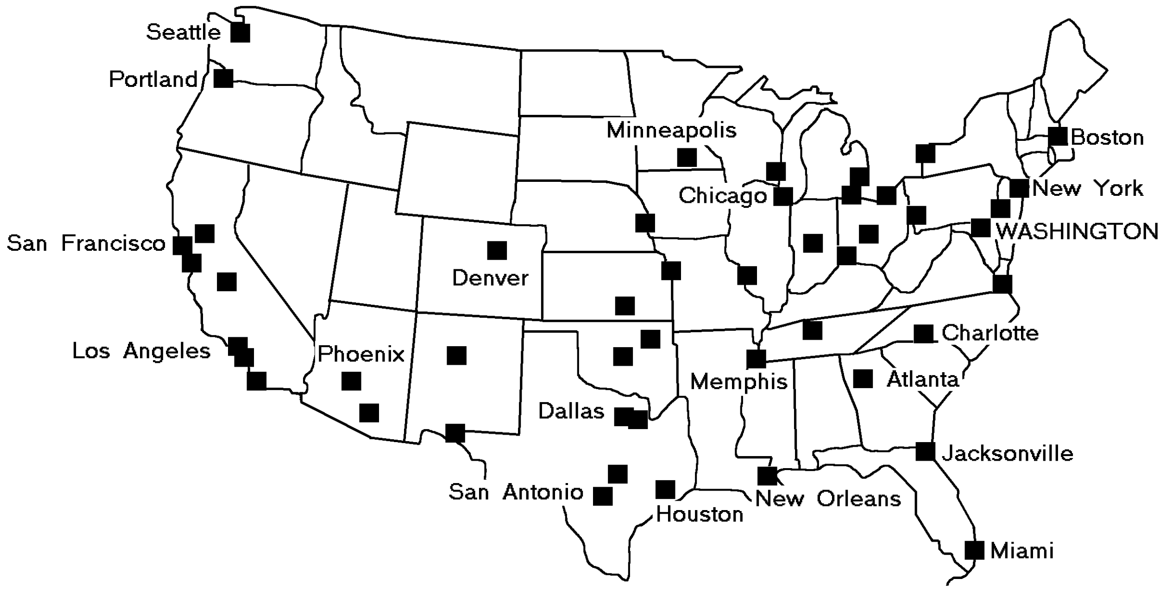


Figure 8-5. Cities used in water balance analysis.

Notice that the winter rain period in which soil moisture is being recharged by the high precipitation which is much greater than ET at that time of the year. The water surplus occurs when the soil cannot store any more water and results in runoff (in natural, undeveloped areas), and coincides with the early spring flood/landslide season in San Francisco. During the late spring and summer, as precipitation becomes almost negligible, available soil moisture is utilized by vegetation for ET purposes. Because the ET is less than PET, there is a deficit that is also shown in Figure 8-4. The deficit is the integral of the PET less the calculated actual ET. This area is calculated month by month in Table 8.2. By comparing Figure 8-4 with the chart for San Francisco in Figure 8-2, it is apparent that the calculations of Mather (1978) and Thornthwaite (1948) have been reproduced.

The amount of the stormwater resource able to be used in each region was plotted in the bar graph shown in Figure 8-6. Most eastern (and western coastal) cities were able to use nearly 100% of the resource. Of course, in using a monthly time step, flooding events are not part of the model. The Rocky Mountains and semi-arid southwest were able to achieve over 90% and the desert southwest (Phoenix) was only able to achieve 24%. Supplemental water would need to be provided in these locations, if reused water is desired to meet irrigation demand, graywater would have to supplement the reused stormwater.

The projected average water deficit for each region are plotted in Figure 8-7. The highest deficit was the desert southwest, with a low rainfall and high PET, followed by the semiarid southwest, then by the Rocky Mountain west, then the northwest,

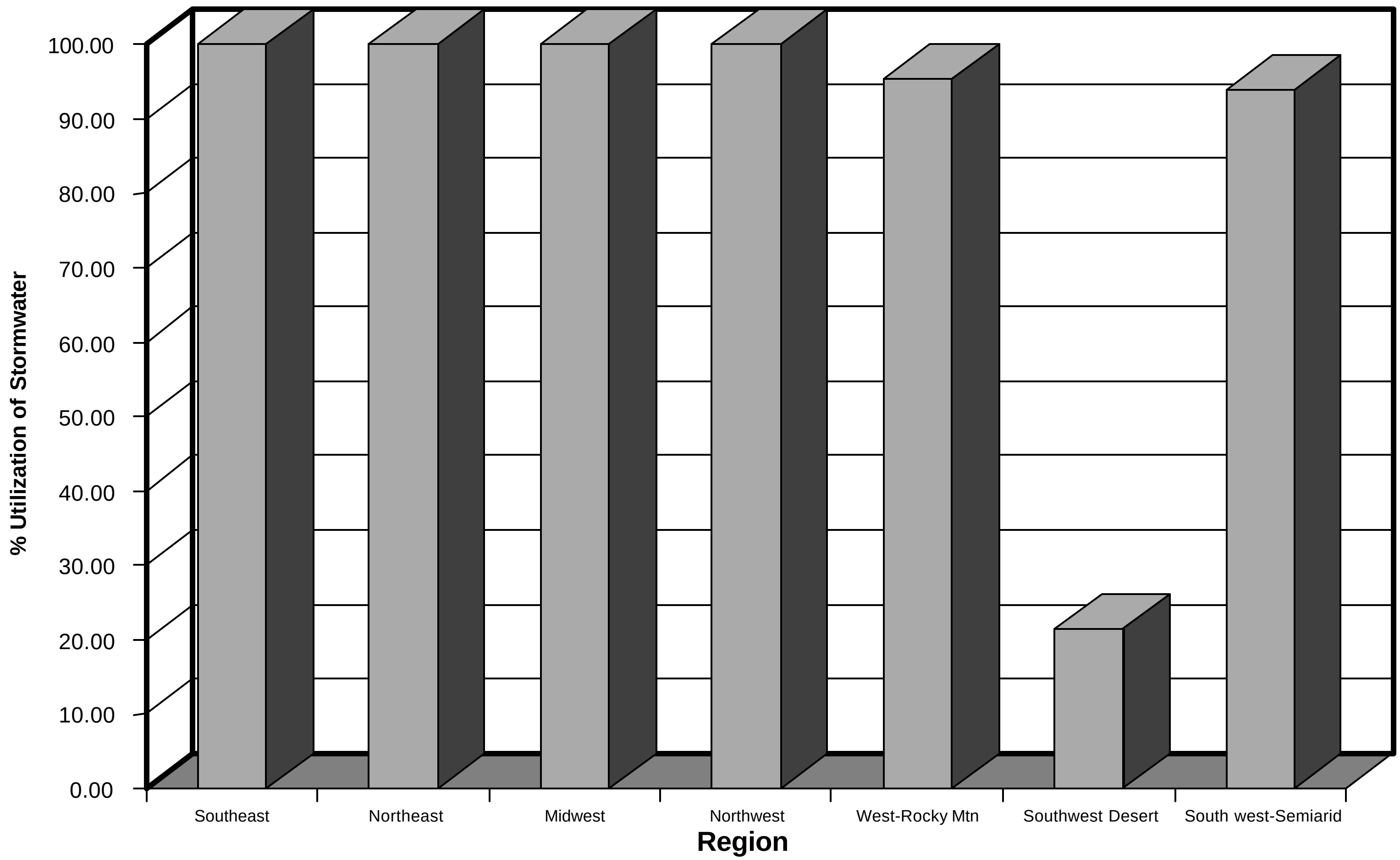


Figure 8-6. Utilization of stormwater by region.

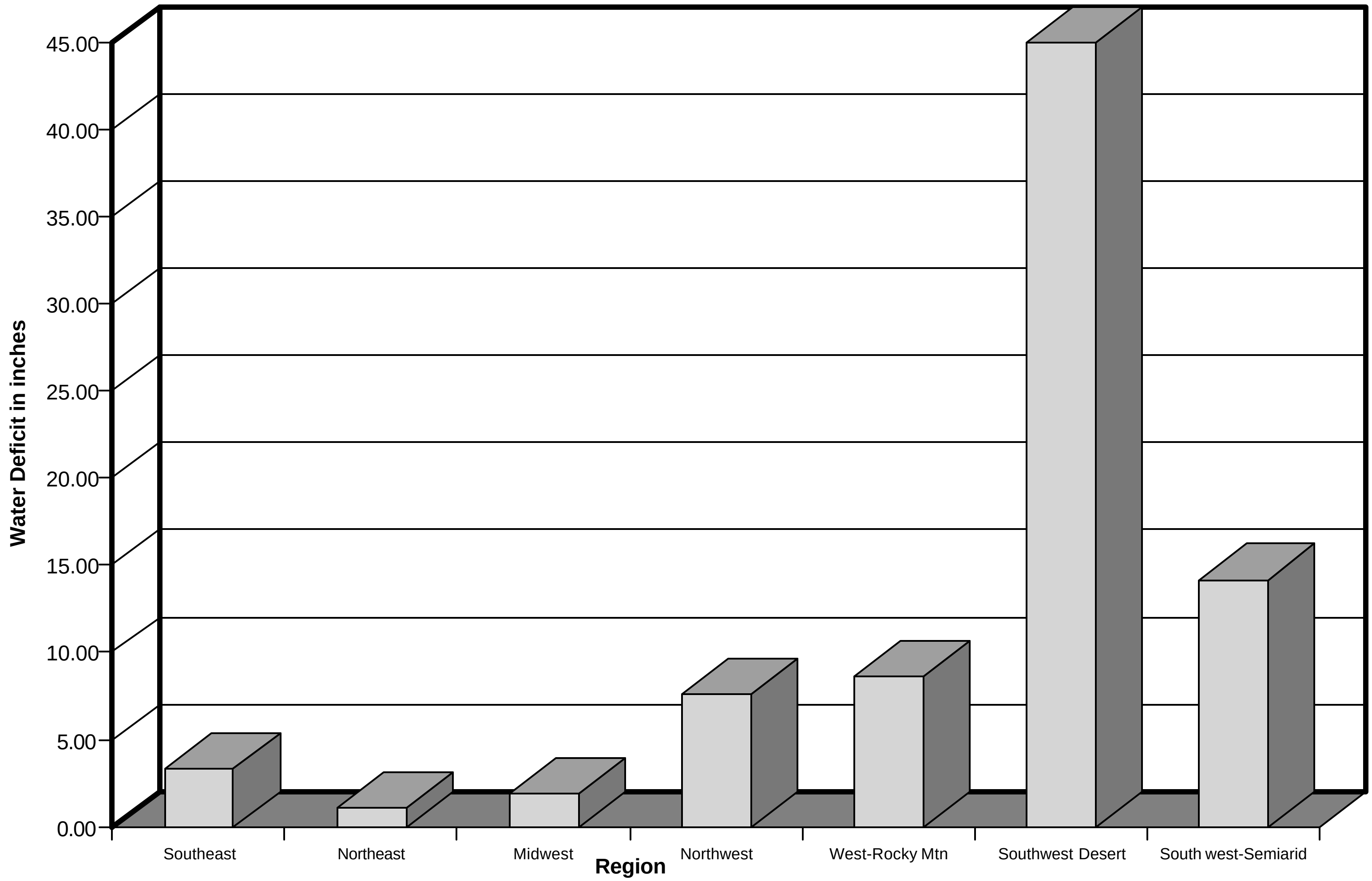


Figure 8-7. Water deficit by region.

southeast, midwest, and northeast.

The annual precipitation, calculated PET, water deficit, and an estimate of the percent error of the Thornthwaite model for each studied city is found in Table 8-3. There may be some variation between these values and other published data depending upon the location of the measurement, as well as the length of the data record. This may affect the error calculation as well. The Thornthwaite model, as stated previously, tends to give better results in non arid areas. The station chosen for Seattle, WA is probably at a higher elevation than published data for the city of Seattle, as the value for precipitation in Table 8-3 is much higher than expected.

The projected storage tank size for each location is plotted in Figure 8-8. San Antonio, TX had the largest tank size, at 25,000 gallons, followed by Dallas, TX at about 17,500 gallons, then Denver, CO at 15,500 gallons. Areas with very dry summers and wet winters such as San Francisco, CA and Los Angeles, CA tended to be next, at around 14,500 gallons. Most areas in the humid east were under 5,000 gallons, except in locations where ET needs outstripped available precipitation, such as in Tampa, FL at 9,000 gallons. The reason very high water deficit areas such as Phoenix, AZ did not result in the largest tanks is that no available storage would have any benefit, that is, the ET needs far exceed available rainfall.

This data compares favorably with Pazwash and Boswell (1997) who found the same nationwide trends when their results are scaled up to the same lot size. They found that the arid southwest tended to require smaller tanks than the rest of the country, due to the lack of available rainfall. Average tank size for other areas ranged from 4320 gallons in the northeast to 6750 in the southeast.

Table 8-3. Summary of annual data for selected stations.

City	State	Annual Precipitation (in)	Annual PET (in)	Annual ET (in)	Annual Water Deficit (in)	Annual Water Surplus (in)	% Error of Model (%)
Atlanta	GA	47.1	37.5	33.4	4.0	13.7	8.10
Boston	MA	47.5	22.3	21.8	0.5	25.6	15.17
Charlotte	NC	43.4	36.8	33.4	3.3	10.0	8.43
Chicago	IL	33.2	26.7	24.1	2.5	9.1	3.73
Dallas	TX	34.6	39.0	30.8	8.2	4.0	25.60
Denver	CO	15.0	23.5	14.9	8.6	0.0	6.76
Houston	TX	45.3	50.0	43.0	7.0	2.3	16.24
Jacksonville	FL	53.3	48.8	48.4	0.5	5.2	19.35
Los Angeles	CA	14.7	39.1	14.8	24.3	0.0	18.16
Memphis	TN	45.7	39.2	32.5	6.7	13.2	7.84
Miami	FL	59.8	57.1	54.3	2.8	6.0	14.21
Minneapolis	MN	24.8	22.3	20.9	1.4	4.3	12.13
New Orleans	LA	63.5	50.4	50.2	0.2	13.3	16.04
New York	NY	42.4	29.1	27.4	1.7	14.9	3.27
Phoenix	AZ	7.2	52.6	7.6	44.9	0.0	15.88
Portland	OR	41.9	25.4	18.9	6.5	23.0	9.71
Salt Lake City	UT	13.9	25.5	13.3	12.2	0.6	8.15
San Antonio	TX	27.9	48.0	27.9	20.1	0.0	13.84
San Francisco	CA	20.8	26.6	16.6	10.1	4.3	3.68
Seattle	WA	64.1	24.1	17.8	6.3	46.3	10.48
Tampa	FL	50.6	52.7	48.8	3.9	1.8	15.21
Washington	DC	40.8	32.2	30.4	1.8	10.4	3.27

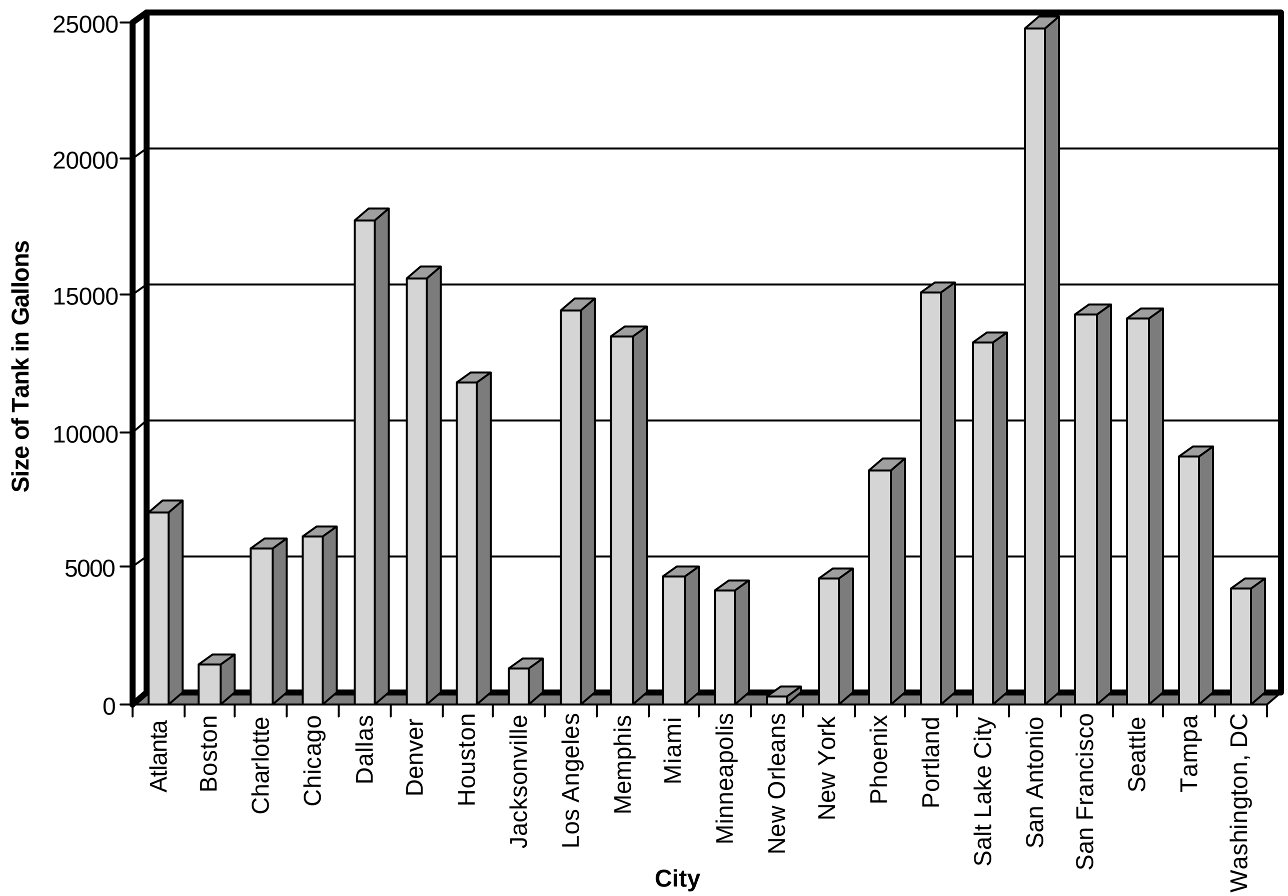


Figure 8-8. Projected residential stormwater storage tank size for studied locations.

Conclusions

In summary, in many areas of the country, particularly in humid areas, enough stormwater can be collected to satisfy average irrigation demands. If driveway areas are eliminated due to possible problems with water quality and ease of collection, the result will be a larger tank size, however, irrigation demand may still be satisfied in a majority of cases. In arid areas, particularly those with high ET requirement, stormwater reuse may not be justified by itself. In these cases, the option of combining storage with treated graywater may be worth considering.

A possible enhancement in the technique could be to apply the model to a daily time series and developing an autoregessive time series model of the PET, ET, and precipitation for each city. Next, a Monte Carlo analysis can be performed to determine that, given the historical data series, a tank sized by this procedure will serve, say, 90% of the ET needs of the parcel. Such an analysis and computer model was developed for rural regions of India by Vyas (1996). An extrapolation of this work to urban/suburban areas of the U.S. needs to be done. In addition, consideration of a daily time step model may be more realistic in this effort. The effect of using several years of data will be to enlarge the tank, as the tank size will increase in order to serve ET needs during more extreme events, such as droughts.

References

Anderson, J.M. (1996a). Current water recycling initiatives in Australia: scenarios for the 21st century. Water Science Technology. (G.B.), 33: 10-11, 37-43.

Anderson, J.M. (1996b). The potential for water recycling in Australia: expanding our horizons. Desalination, 106: 1-3, 151-156.

Asano, T., and A. D. Levine (1996). Wastewater reclamation, recycling and reuse: past, present, and future. Water Science Technology. (G.B.), 33: 10-11, 1-13.

Ashcraft, J. G., and M. G. Hoover (1991). Water reuse--implementation and costs in southern California. In Water Supply and Water Reuse: 1991 and Beyond. Proceedings of the AWRA Conference. Denver, CO.

Butler, D. and J. Parkinson (1997). Towards sustainable urban drainage. Water Science and Technology. 35 (9), 53-63.

Courtney, B.A. (1997). An Integrated Approach to Urban Irrigation: The Role of Shading, Scheduling, and Directly Connected Imperviousness. MS Thesis. Department of Civil, Environmental, and Architectural Engineering. University of Colorado. Boulder, CO.

Field, R. (1993). Reclamation of Urban Stormwater. Integrated Stormwater Management. Field, R., M.L. O'Shea, and K.K. Chin (Ed.), p.307.

Foster, K. E., M. M. Karpiscak, and, R. G. Brittain (1988). Casa Del Agua: a residential water conservation and reuse demonstration project in Tucson, AZ. Water Resources Bulletin. 24(6):1201-1206.

Grimmond, C. S. B. and T. R. Oke (1986). Urban water balance 2: Results from a suburb of Vancouver, British Columbia. Water Resources Research. 22(10):1404-1412.

Grimmond, C. S. B., T. R Oke, and D. G. Steyn (1986a). Urban water balance 1: a model for daily totals. Water Resources Research. 22(10):1397-1403.

Haarhoff, J., and B. Van der Merwe (1996). Twenty-five years of wastewater reclamation in Windhoek, Namibia. Water Science Technology. (G.B.), 33: 10-11, 25-35.

Harper, G. (1991). Reuse of Stormwater: Design Curves for Florida. MS Thesis. University of Central Florida. Orlando, FL.

Harrison, T.J. (1993). Stormwater Reuse Design Curves for Southwest Florida Water Management District. In SWFMD. Proceedings of 3rd Biennial Stormwater Research Conference. Brooksville, FL.

Heaney, J.P. and L.T. Wright (1997). On Integrating Continuous Simulation and Statistical Methods for Evaluating Urban Stormwater Systems. Chapter 3 in James, W. (Editor). Advances in Modeling the Management of Stormwater Impacts. Vol. 5. CHI. Guelph, ON, Canada. P. 44-76.

Heaney, J.P., L.T. Wright, D. Sample, B. Urbonas, B. Mack, M. Schmidt, M. Solberg, J. Jones, J. Clary, and T. Brown (1998). Development of Methodologies for the Design of Integrated Wet-Weather Flow Collection/Control/Treatment Systems for Newly Urbanizing Areas. Draft Rep. to U.S. EPA. Natl. Risk Manage. Res. Lab. Cincinnati, OH.

Herrmann, T., and K. Hase (1996). Way to get water rainwater utilization or long-distance water supply? A holistic assessment. In Proc. of the 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Herrmann, T., U. Schmida, U. Klaus, and V. Huhn (1996). Rainwater utilization as component of urban drainage schemes: hydraulic aspects and pollutant retention. In Proc. 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Imbe, M., T. Ohta, and N. Takano (1996). Quantitative assessment of improvement in hydrological water cycle in urbanized river basin. In Proc. of the 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Karpiscak, M. M., K. E. Foster, and N. Schmidt (1990). Residential water conservation: Casa Del Agua. Water Resources Research. 26: 6, 939-948.

Law, I. B. (1996). Rouse Hill—Australia's first full scale domestic non-potable reuse application. Water Science Technology. (G.B.), 33: 10-11, 71-78.

Leemans, R. (1988). Dataset developed during the YSSP summary program 1988, at IIASA. Laxenburg, Austria for the BIOSPHERE.

Lejano, R. P., F. A. Grant, T. G. Richardson, B. M. Smith, and F. Farhang (1992). Assessing the benefits of water reuse: applying a cost-benefit allocation procedure. Water Environment & Technology. 4 (8), 44-50.

Mallory, C. W., and J. J. Boland (1970). A system study of storm runoff problems in a new town. Water Resources Bulletin. 6 (6), 980-989.

Mangeot, E.F. (1996). City of Boulder Wastewater Treatment Plant Model Stormwater Surge Analysis Using GPX-X. Independent Study for Professor Silverstein. Department of Civil, Environmental, and Architectural Engineering. University of Colorado. Boulder, CO.

Mather, J. R.(1978). The Climatic Water Budget in Environmental Analysis. D. C. Heath and Company. ISBN: 0-669-02087-7.

Mitchell, V.G., R.G. Mein, and T.A. McMahon (1996). Evaluating the resource potential of stormwater and wastewater: an Australian perspective. In Proc. of the 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Nelen, A.J.M., A.C. de Ridder, and E.C. Hartman (1996). Planning of a new urban area in a municipality of Ede using a new approach to environmental protection. In Proc. of the 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Nowakowska-Blaszczyk, A., and J. Zakrzewski (1996). The sources and phases of increase of pollution in runoff waters in route to receiving waters. In Proc. 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Oron, G. (1996). Management modeling of integrative wastewater treatment and reuse systems, wastewater reclamation and reuse 1995. Water Science Technology. (G.B.), 33: 10-11, 95-105.

Pazwash, H., and S. Boswell (1997). Management of roof runoff conservation and reuse. Proc. 24th ASCE Water Resources Planning and Management Annual Conf., Aesthetics on the Constructed Environment. Houston, TX. 784.

Pisano, W.C. and H. Brombach (1997). Solids Settling Curves. Water Environment Technology. Vol. 8, No.4, P 27-33.

Pitt, R., R. Field, M. Lalor, and M. Brown (1996). Groundwater Contamination from Stormwater Infiltration. Chelsea, MI. Ann Arbor Press.

Pruel, H.C. (1996). Combined sewage prevention system (CSPS) for domestic wastewater source control. In Proc. of the 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Requa, A., J. M. Kelly, J. A. Burgh, and M. Manzione (1991). Successful implementation of an urban landscaping recycled water program. In Water Supply and Water Reuse: 1991 and Beyond. Proceedings of the Conference of the AWRA. Denver, CO.

Tchobanoglous, G., and A. Angelakis (1996). Technologies for wastewater treatment appropriate for reuse potential for application in Greece. Water Science Technology. (G.B.), 33: 10-11, 15-24.

Thornthwaite, C. W. (1948). An approach toward a rational classification of climate. Geographical Review. volume 38 pp. 55-94.

Tselentis, Y., and S. Alexopoulou (1996). Effluent reuse options in Athens metropolitan area: a case study. Water Science Technology. (G.B.), 33: 10-11, 127-137.

U.S. Geological Survey (1970). The National Atlas of the United States of America. Reston, VA.

Veldkamp. R. G., T. Hermann, V. Colandini, L. Terwel, and G. D. Geldof (1997). A decision network for urban water management. Water Science Technology. (G.B.) 36: 8-9, 111-115.

Vorosmarty, C. J. and B. Moore (1991). Modeling basin-scale hydrology in support of physical climate and global biogeochemical studies: an example using the Zambezi River. Surveys in Geophysics. volume 12, pp. 271-311.

Vorosmarty, C. J., C. J. Willmott, B. J. Choudhury, A. L. Schloss, T. K. Stearns, S. M. Robeson, and T. J. Dorman (1996). Analyzing the discharge regime of a large tropical river through remote sensing, ground-based climatic data and modeling. Water Resources Research. 32(10):3137-3150.

Vorsomarty, C. J., B. Moore, A. L. Grace, M. P. Gildea, J. M. Melillo, B. J. Peterson, E. B. Rastetter, and P. A. Steudler (1989). Continental scale models of water balance and fluvial transport: an application to South America. Global Biogeochemical Cycles. 3(3):241-265.

Vyas, V. (1996). Monte-Carlo simulation of rainwater harvesting systems. Raindrop. December, 1996 and web page http://www.geocities.com/RainForest/Canopy/4805/FAQ.html.

Wanielista, M. (1993). Stormwater Reuse: An Alternative Method of Infiltration. In Proc. National Conf. on Urban Runoff Management: Enhancing Urban Watershed Management at the Local, County, and State Levels. U.S. EPA. Cincinnati, OH.

Willmott, C. J. (1977). WATBUG: A FORTRAN IV Algorithm for Calculating Climatic Water Budget. Water Resources Center. Delaware University; Contribution Number 23, Report Number 1. Publications in Climatology Vol. XXX, No. 2.

Yates, D. N. (1996). WatBal: an integrated water balance model for climate impact assessment of river basin runoff. Water Resources Development. 12(2):121-139.

Chapter 9

Urban Stormwater and Watershed Management: A Case Study

James P. Heaney, Len Wright, and David Sample

Overview

Interest in watershed management has waxed and waned over the past century. The concept of integrated water and land management was first articulated in the western U.S. by John Wesley Powell in a report to the Congress in 1878 (Peterson 1984). However, Congress rejected his idea and continued to use an ad hoc approach to authorizing projects. During the 20th century, interest in watershed planning has come and gone several times. Following World War I, unified planning at the river basin scale flourished with major studies and implementation on numerous river basins, (e.g., the creation of the Tennessee Valley Authority). The National Resources Planning Board provided the leadership for these efforts (Viessman and Welty 1985). Increased environmental awareness during the 1960's and 1970's led to expanded efforts to evaluate water quality and related problems on a regional level. During the 1980's, primary reliance was placed on a command and control approach for addressing water resources problems. A strong move back to the watershed management approach began a few years ago, (e.g., see the Proceedings of Watershed 93 and Watershed 96, WEF, 1993, 1996). While it is axiomatic that integrated, holistic, sustainable infrastructure systems are very desirable, demonstrated success stories of how such systems might function effectively are rare (Heaney 1993).

Watershed Planning Methodologies

Early watershed planning efforts focused on developing "master plans" which, once approved, would serve as a blueprint for management in the basin. Prior to computers, such efforts faced severe technological limitations in bringing together large amounts of information and analyzing alternatives in a systematic manner. The widespread availability of mainframe computers in the 1960's and associated computer-based simulation and optimization techniques led to large-scale efforts to develop "rational" master plans (Maass et al. 1962). Integrated river basin planning models were developed as early as 1971. An updated summary of these quantitative methodologies is contained in Mays and Tung (1992) and Wurbs (1994). The thrust in developing better planning methodologies was in devising ever-more complex models, (e.g., three dimensional lake models, nonlinear programming models). Unfortunately, the sophistication of the models greatly outstripped the availability of data. Nevertheless, models have had a strong positive influence in water resources planning (Office of Technology Assessment 1982).

Dissatisfaction with rational planning models and major improvements in metrology led to the more recent shift to data rather than model driven approaches wherein the analyst attempts to match the models with the data. These information driven approaches are often classified as Decision Support Systems (DSS) (Loucks 1995). Contemporary DSS's contain a mixture of simulation and optimization models, databases, geographical information systems, typically with a graphics front-end to integrate these systems. The DSS should incorporate real-time control systems if they have been installed. The DSS is more than a series of interfaced programs. It also embodies a different philosophy of planning. Rather than focusing on "solving" the "problem", the DSS provides an operational framework in which continuous process improvement is stressed.

Contemporary Principles of Watershed Management

During recent years, several national and regional groups have articulated new principles of water and environmental management. A summary of these positions follows.

American Water Resources Association

The American Water Resources Association (AWRA) represents the largest collection of professionals dealing with water resources problems. They published the following list of seven guiding principles of water resources management (Anonymous 1992):

1. Water problems should be approached in a holistic way with the watershed as the basic planning unit; and the water requirements of natural systems within the watershed must be fully integrated into water-management decisions.

2. The framework for policy making must be flexible and adaptive to changing conditions, needs, and values, yet provide a level of predictability and timeliness needed to support management and investment decisions; management strategies must focus on appropriate geography to effectively deal with the problems at hand; and the public must understand the nature of the problems and how resource managers intend to solve them.

3. The States play a key role in water management and should be delegated responsibility for specific water-related Federal programs; authority and accountability should be decentralized to the lowest capable level of government while ensuring oversight and enforcement of these programs; obstacles to meaningful intergovernmental partnerships, such as overlapping missions, jurisdictional boundaries, and responsibilities, must be overcome.

4. Water policy development should express a preference for negotiation, market-like approaches, and performance standards and should include more consultation, cooperation, and concurrence between all levels of government and non-governmental entities with interests in the policies.

5. Federal, State, and local participation should be encouraged in the development of each other's program policy development, implementation, and administration; more leadership capacity needs to be developed among politicians, water professionals, and the public to champion concerns and reforms.

6. Freshwater is a fundamental integrating ingredient in natural resources management and an essential building block for a competitive and healthy economy.

7. The goal of freshwater sustainability should be a guiding principle for future water-resource management.

Water Environment Federation

The WEF is the professional organization, which represents the water quality field. They have been conducting a major initiative called Water Quality 2000. The output of the third phase of their effort is the result of an 18-month consensus process that included more than 100 experts representing a wide variety of interests. This report calls for a national water policy that will improve protection of surface and ground waters by combining the following three interrelated strategies (WEF 1993):

1. Pollution prevention.
2. Increased individual and collective responsibility for protecting water resources.
3. Reorientation of water research programs and institutions along natural watershed boundaries.

U.S. Environmental Protection Agency

The U.S. EPA has adopted a watershed approach to water quality management (US EPA, 1991). This posture represents a revisiting of their earlier leanings in this direction.

Case Study of Urban Stormwater Management within a Watershed Framework

Introduction

The benefits and challenges of using an integrated, watershed-based approach to water and environmental management can be demonstrated using a case study with meaningful data and models. BCW, which includes the City of Boulder, was selected for this purpose. A map of BCW is shown in Figure 9-1. BCW is a textbook watershed with its origins in the Rocky Mountains from where it flows out of the mountains through the Front Range of Colorado.

With the beginning of mining in 1858, the water and land associated with development activities have had a significant impact on BCW. The initial mining activities altered streamflows, greatly increased erosion and pollution, and forever altered the "natural" hydrology. From 1858 to the present, BCW has been drastically altered by activities such as mining, urbanization, agricultural activities, and hydropower development. BCW suffered serious stormwater pollution from mining activities beginning in the 1860s. Thus, nonpoint pollution is an old problem in BCW.

BCW has also been adapted to provide water supply, flood control, recreation, and instream flow needs. These interventions are both structural and nonstructural. Structural interventions include construction of reservoirs, canals, pipelines, pump stations, hydropower generation, water and wastewater collection and treatment systems, flood control levees, instream and wetland restoration, and imports and exports of water. Nonstructural interventions include flood warning systems, floodplain management, water rights enforcement, water conservation programs, and education about watershed protection.

The end result of all of these interventions is a complex watershed system, which has been adapted to serve the needs of society as well as the natural system. This level of development and adaptation is typical of watersheds in the U.S. and other developed areas. Dealing with the watershed as a system is essential in contrast with trying to isolate one component of it and assume away all of the complexity that is associated with this system. While the focus of this report is urban stormwater quality management, these other considerations should also be kept in mind. The components of BCW are discussed in the following sections.

Hydrology

Introduction

BCW can be partitioned into three main sources: North Boulder Creek, Middle Boulder Creek, and South Boulder Creek, as shown in Figure 9-1. According to WBLA, Inc. (1988), the general water budget for the system inflows, under natural conditions, is as follows:

Source	Percent of Total
North Boulder Creek	20
Middle Boulder Creek	30
South Boulder Creek	40
Other Tributaries	10
Total	100

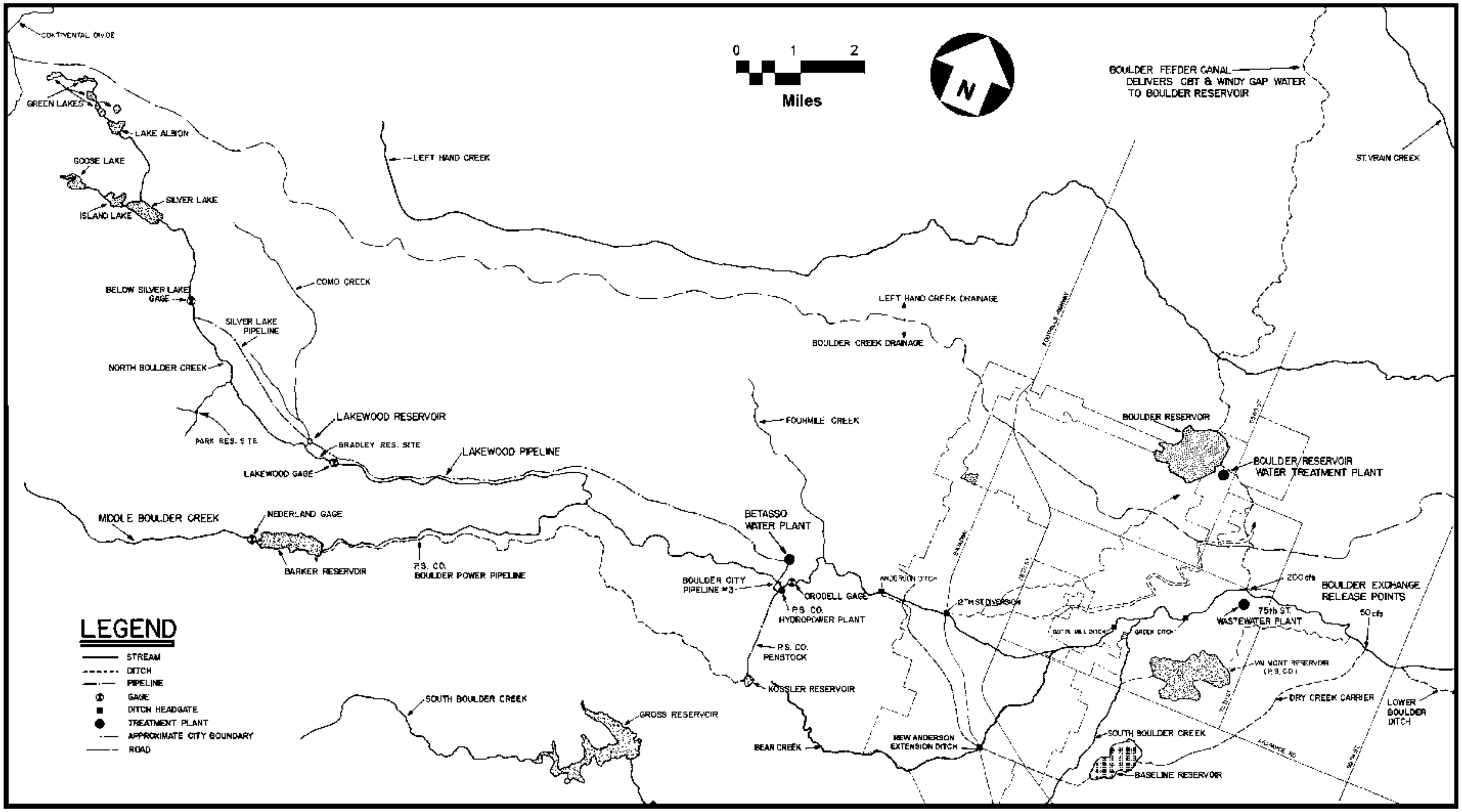


Figure 9-1. Boulder Creek Watershed, CO. City of Boulder 1998. (Reprinted Courtesy of Hydrosphere)

The total estimated natural inflow averaged 140,000 acre feet per year. The natural inflow is estimated by correcting the observed historical inflows for development activities such as storage, imports, and exports. The reconstructed expected natural inflows of Boulder Creek at Broadway, which is located at the upstream end of the City of Boulder, are shown in Table 9-1 and Figure 9-2.

The natural inflow averages 108 cfs. Depletions have reduced this natural flow to an average of 52 cfs, or 48% of the natural inflow. The monthly pattern of inflows, shown in Table 9-1 and Figure 9-2, indicates the dominant influence of the spring runoff in supplying water to the downstream portion of BCW. About 72 % of the annual runoff occurs during May, June, and July. The traditional low flow period of concern for water quality management occurs in late summer when the stream temperatures are high and flow in the receiving water is low. The lowest historical flows occur in October at the end of the irrigation season as shown in Table 9-1. The average flow at Broadway in October is 10 cfs. However, these inflows at Broadway do not necessarily pass through the city. Much of this inflow is diverted between Broadway and 75th St., the downstream end of the City of Boulder.

Table 9-1. Boulder Creek watershed streamflows on Main Boulder Creek below Broadway in Boulder, CO (WBLA Associates 1988).

Month	Natural (cfs)	Historical (cfs)	Natural (%)	Historical (%)
January	15	33	1.2	2.5
February	18	33	1.4	2.5
March	22	22	1.7	1.7
April	58	35	4.5	2.7
May	250	115	19.3	8.9
June	435	180	33.5	13.9
July	252	90	19.4	6.9
August	105	25	8.1	1.9
September	52	20	4.0	1.5
October	40	10	3.1	0.8
November	30	25	2.3	1.9
December	20	35	1.5	2.7
Avg.	**108**	**52**	**100**	**48.0**

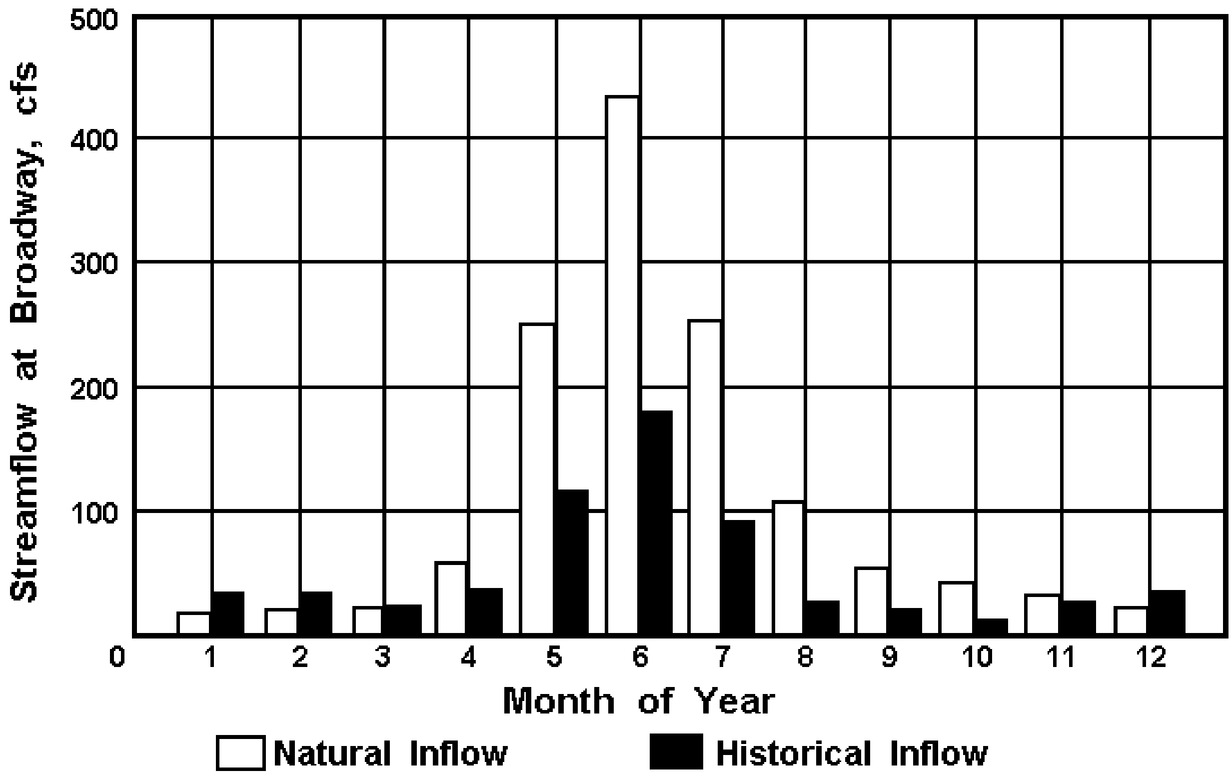


Figure 9-2. Monthly inflows of Boulder Creek to Boulder, CO.

Precipitation Analysis

The average annual precipitation in Boulder is 18.2 inches with about two thirds of this occurring between April and September. Total annual precipitation has ranged from 10 to 28 inches. Annual and monthly total precipitation data are presented in Figures 9-3 and 9-4 and Table 9-2. May is the wettest month of the year.

Storm event statistics were tabulated using NWS hourly rainfall data. A storm event is defined as ending when it hasn't rained for six consecutive hours. An estimated minimum storm event precipitation of 0.15 inches is needed to initiate runoff. The relative frequency distribution for these runoff producing events (RPE) is shown in Figure 9-5. An average of 29.27 RPEs occur per year. The monthly distributions of storm events is shown in Table 9-3 and Figures 9-6 to 9-9. An average of 2.44 RPEs occur per month with as little as 1.3 RPEs in January to a high of 3.6 RPEs in May. The average RPE volume/month is 1.25 inches. The mean volume per RPE is 0.49 inches. The mean event duration is 5.8 hours and the mean interevent time is 318 hours. Overall, RPEs occur less than 2% of the year. For Boulder, the precipitation falling from November to March is typically occurs as snow.

Streamflow Stations

A summary of available stream gauging stations is presented in Table 9.4. A brief summary of the individual watersheds and stream gauging stations follows.

North Boulder Creek

The flows in North Boulder Creek are directly affected by seven city owned reservoirs with a total storage capacity of about 7,000 acre feet (WBLA 1988). The City diverts water from North Boulder Creek via the Silver Lake and Lakewood pipelines. Natural flows at Lakewood average 21,800 acre feet per year over about 31 square miles of drainage or about 0.97 cfs/mi^2. As shown in Table 9-1, development has had a major impact on North Boulder Creek with a combination of storage and direct diversions. The natural flow below Lakewood of 31.25 cfs has been reduced by about one third due to man's activities with no flow in the stream during the colder months of the year. No long-term stream gauging stations exist for North Boulder Creek. The only available record is a few years of data on the upper parts of the North Boulder Creek Watershed. Flows in North Boulder Creek are affected by upstream storage and a major diversion of water for the City of Boulder's water supply system via the Lakewood pipeline. Natural flows for North Boulder Creek can be estimated based on its hydrologic similarity to Middle Boulder Creek above Nederland.

Middle Boulder Creek

According to WBLA (1988), Middle Boulder Creek flows essentially undisturbed into Barker Reservoir at Nederland. The average runoff is about 1.55 cfs/mi^2. Barker Dam and associated diversions for water supply and hydropower exert a drastic influence on Middle Boulder Creek downstream of Barker Dam. The City diverts water for water supply and Public Service Company of Colorado diverts water for hydropower, both via the Barker pipeline. As shown in Table 9-1, the natural outflow has decreased from about 108 cfs to less than 52 cfs, a loss of over half of the natural flow in the stream. With current diversions, only about one or two cfs of flow reach the confluence of North Boulder Creek and Middle Boulder Creek during the colder months of the year.

The flows of Middle Boulder Creek as it enters the City are dominated by PSCO hydropower releases and diversions by a large number of agricultural ditches. Historically, during dry years, extended periods of flows less than one cfs have been experienced below Broadway due to agricultural diversions and Boulder's exchange operations (WBLA 1988). Winter flows fluctuate wildly due to hydropower releases with flows ranging from 2 to 140 cfs over a single day as shown in Figure 9-10.

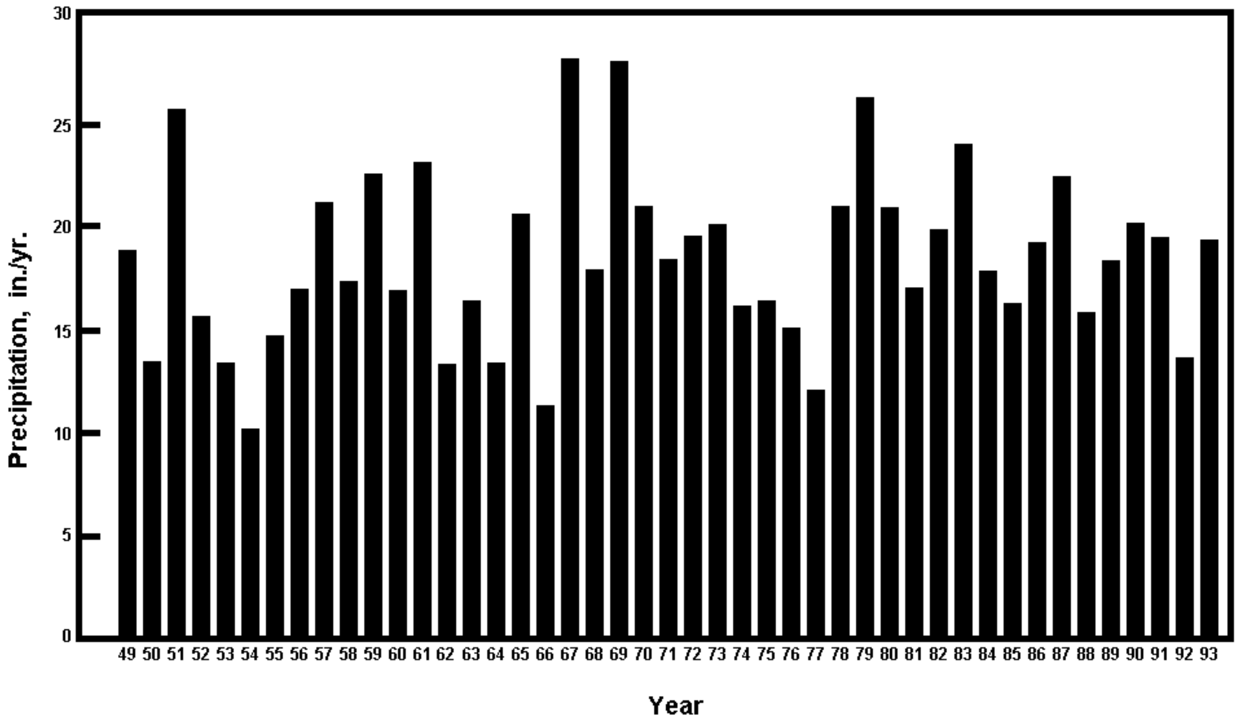


Figure 9-3. Mean annual precipitation in Boulder, CO.

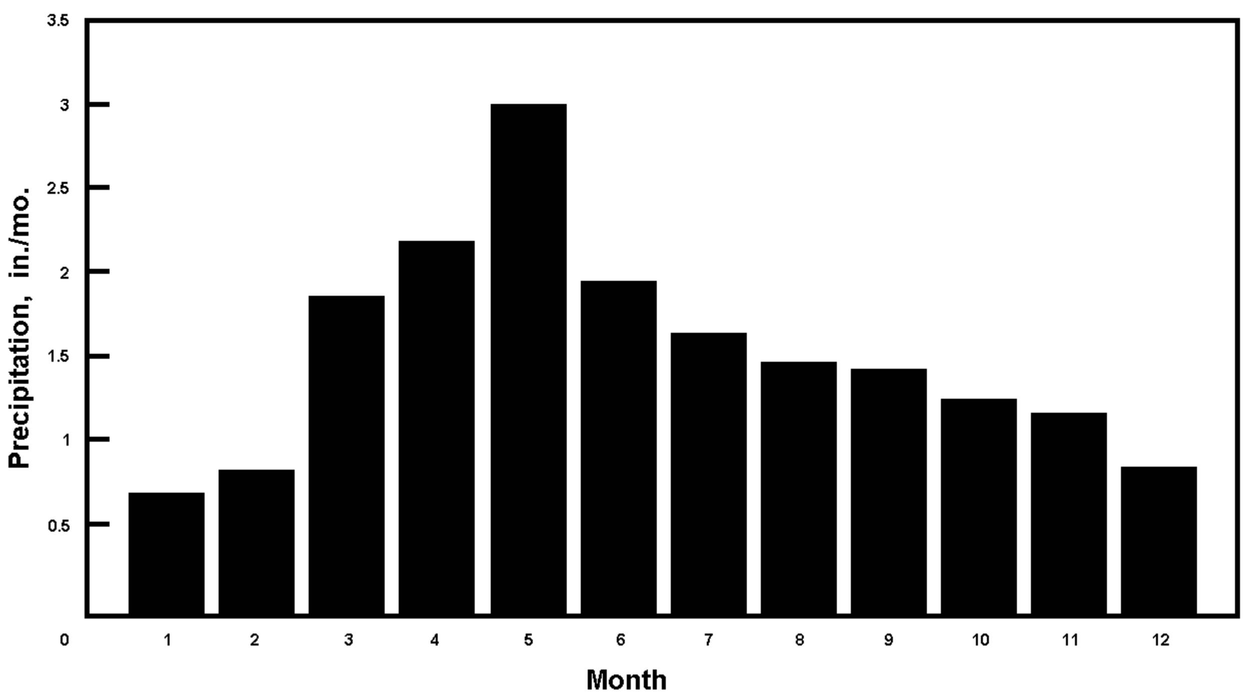


Figure 9-4. Mean monthly precipitation in Boulder, CO.

Table 9-2. Monthly precipitation in Boulder, CO, 1949-1993.

	Month												
Yr.	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
49	0.52	0.10	2.47	2.12	3.28	7.03	1.05	0.31	0.00	1.49	0.00	0.27	18.6
50	0.92	0.24	0.34	2.74	3.07	0.72	1.47	0.19	1.30	0.38	1.79	0.27	13.4
51	0.67	0.93	1.97	2.23	2.01	2.09	1.16	8.59	0.88	2.62	1.12	1.40	25.7
52	0.03	0.39	1.71	2.84	3.73	0.93	0.64	3.47	0.29	0.24	1.29	0.00	15.6
53	0.22	0.66	1.60	2.18	2.13	0.76	2.26	0.92	0.00	0.51	1.03	1.04	13.3
54	0.57	0.20	1.28	0.88	1.08	0.97	1.79	0.44	1.31	0.34	0.64	0.65	10.2
55	0.32	1.27	2.03	0.20	2.25	1.99	0.85	2.25	0.80	0.37	1.42	0.90	14.7
56	0.24	1.70	1.30	1.44	2.85	2.00	2.78	1.53	0.00	0.48	1.83	0.71	16.9
57	0.85	0.99	0.56	3.12	8.61	0.46	0.73	2.35	0.80	1.86	0.69	0.06	21.1
58	0.70	0.35	2.88	2.74	3.91	1.38	1.35	0.67	0.74	0.61	0.99	0.88	17.2
59	1.37	1.59	2.65	3.71	3.62	0.51	0.56	1.02	3.39	2.66	1.12	0.14	22.3
60	0.68	1.81	1.13	2.13	3.68	0.52	0.94	0.26	0.52	2.76	0.66	1.71	16.8
61	0.75	1.04	3.48	1.39	3.37	2.11	1.69	1.65	4.47	1.25	1.13	0.69	23.0
62	1.87	1.15	0.64	0.90	2.06	2.49	1.45	0.21	0.24	1.27	0.70	0.17	13.2
63	1.00	0.53	2.45	0.17	1.05	4.58	0.46	1.84	2.35	0.35	0.72	0.83	16.3
64	0.39	0.96	1.59	1.41	2.06	1.58	2.20	0.31	0.34	0.22	1.17	1.00	13.2
65	1.11	1.73	2.10	2.38	1.34	2.55	4.81	0.33	3.00	0.24	0.25	0.66	20.5
66	0.21	1.27	0.26	1.44	0.70	1.27	0.90	0.45	2.94	0.79	0.60	0.30	11.1
67	0.84	0.61	1.29	1.90	5.00	4.83	2.81	4.94	0.92	1.29	1.46	2.07	28.0
68	0.20	1.20	0.86	2.27	2.33	2.54	1.30	3.84	1.26	0.47	0.81	0.65	17.7
69	0.36	0.35	1.01	1.05	8.51	5.24	2.33	0.46	0.47	6.36	0.96	0.72	27.8
70	0.15	0.82	5.72	1.25	1.07	2.68	1.34	0.17	4.31	1.25	1.50	0.50	20.8
71	0.70	2.10	1.10	5.40	1.00	0.10	1.00	0.20	4.30	0.90	0.80	0.60	18.2
72	1.40	0.70	1.00	1.30	2.99	2.30	2.40	1.20	1.00	1.30	2.50	1.30	19.4
73	1.40	0.20	1.70	5.50	4.00	0.50	1.10	0.20	1.43	0.70	1.70	1.40	19.8
74	1.00	1.20	1.50	2.70	0.00	2.40	0.80	0.60	1.90	2.10	1.30	0.50	16.0
75	0.50	1.10	2.00	2.80	2.99	1.60	0.40	0.90	1.00	0.80	1.40	0.70	16.2
76	0.60	0.40	1.60	2.10	1.40	1.20	1.80	1.10	2.80	1.20	0.30	0.40	14.9
77	0.20	0.70	0.50	3.10	0.60	0.50	3.10	1.90	0.20	0.30	0.50	0.20	11.8
78	0.80	0.40	1.60	3.00	7.00	1.11	1.00	1.30	0.10	2.10	0.20	2.10	20.7
79	0.70	0.30	2.70	2.10	5.40	3.00	0.70	3.90	0.50	1.30	3.00	2.40	26.0
80	1.50	1.00	2.60	5.50	3.80	0.20	1.70	1.10	1.20	0.80	1.10	0.20	20.7
81	0.20	0.40	2.30	1.30	4.80	1.50	1.70	1.10	0.80	1.20	0.30	1.20	16.8
82	0.20	0.82	0.60	0.50	4.50	2.20	4.60	1.50	1.43	1.20	0.40	1.60	19.5
83	0.20	0.10	4.70	3.00	4.70	2.30	2.60	0.80	0.30	0.20	3.90	0.90	23.7
84	0.50	0.90	2.60	0.00	2.80	1.60	1.60	2.00	0.90	4.00	0.00	0.60	17.5
85	0.70	1.00	1.40	1.90	1.20	1.80	1.90	0.00	2.50	0.90	1.70	1.00	16.0
86	0.10	1.00	0.60	4.80	2.50	1.50	1.70	0.20	0.80	3.40	1.90	0.50	19.0
87	1.10	0.82	2.20	2.30	1.80	5.70	1.10	1.80	1.00	0.80	1.70	1.90	22.2
88	0.40	1.10	2.40	1.40	3.40	0.60	0.50	1.20	1.90	0.10	0.70	1.80	15.5
89	0.70	1.00	0.90	1.80	3.00	2.10	1.30	1.40	2.90	1.20	0.30	1.50	18.1
90	0.90	0.70	4.40	2.20	1.70	0.20	3.20	1.80	1.80	0.80	1.40	0.80	19.9
91	1.00	0.10	0.50	2.00	4.10	1.80	2.70	1.50	1.50	0.80	3.20	0.00	19.2
92	0.70	0.00	3.40	0.50	1.90	1.00	1.10	3.20	0.00	0.40	0.30	0.86	13.4
93	0.67	0.82	1.40	2.10	1.20	2.90	0.70	0.60	3.70	2.22	2.20	0.60	19.1
Mean	0.67	0.82	1.84	2.17	2.99	1.94	1.63	1.46	1.43	1.26	1.17	0.86	18.24
Max.	1.84	2.10	5.72	5.50	8.61	7.03	4.81	8.59	4.47	6.36	3.90	2.40	28.00
Min.	0.03	0.00	0.26	0.00	0.00	0.10	0.40	0.00	0.00	0.10	0.00	0.00	10.20
STD	0.42	0.49	1.16	1.29	1.87	1.50	0.99	1.55	1.24	1.17	0.83	0.60	4.15
C of V	0.62	0.60	0.63	0.59	0.63	0.77	0.61	1.06	0.87	0.93	0.71	0.69	0.23

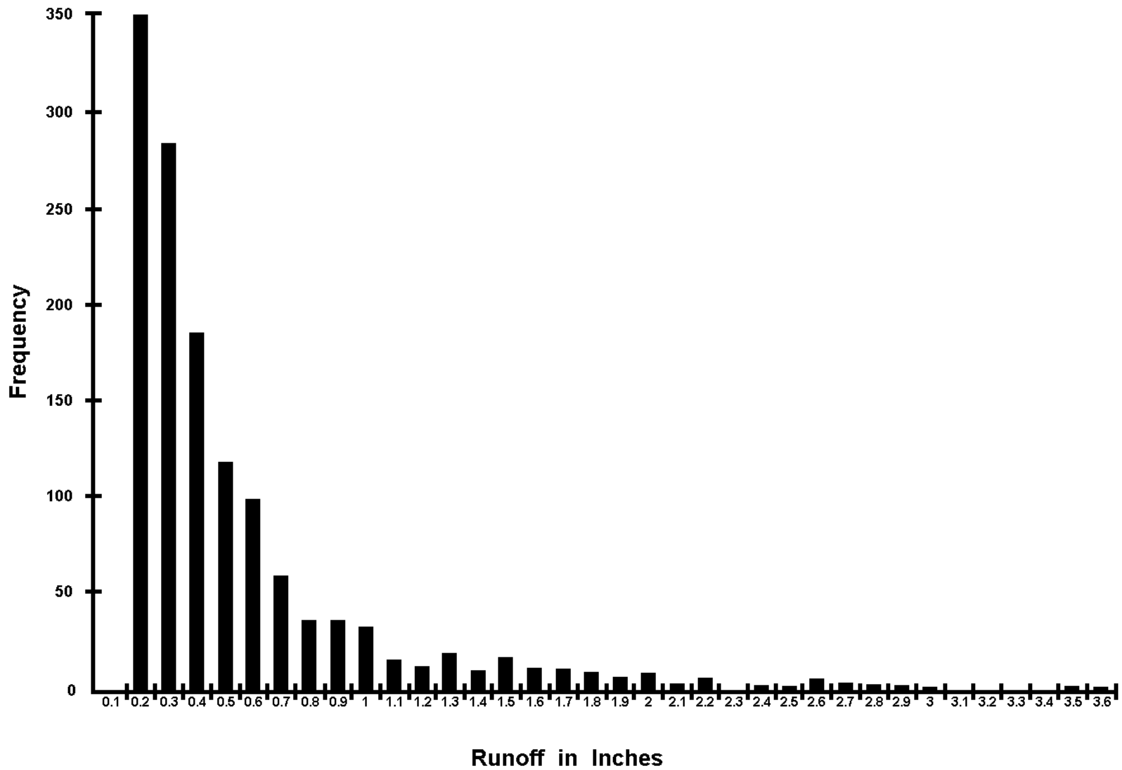


Figure 9-5. Relative frequency for runoff producing events in Boulder, CO.

Table 9-3. Summary of monthly and annual storm event statistics for Boulder, CO 1949-1993.

Month	Events/mo.	Volume				Duration			Interevent Time		
		Average (in./mo.)	Mean (in./event)	STD (in./event	C of V	Mean (hours)	STD (hours)	C of V	Mean (hours)	STD (hours)	C of V
Jan	1.31	0.49	0.372	0.216	0.58	6.68	5.212	0.781	546	455	0.834
Feb	1.47	0.60	0.407	0.210	0.52	6.68	6.885	1.031	462	444	0.961
Mar	3.13	1.54	0.490	0.414	0.85	6.04	6.460	1.070	319	428	1.341
Apr	3.20	1.84	0.574	0.482	0.84	6.69	5.308	0.794	206	179	0.872
May	3.58	2.48	0.693	0.822	1.19	7.71	9.192	1.192	211	263	1.248
Jun	2.84	1.68	0.592	0.612	1.03	5.70	6.125	1.074	199	231	1.162
July	3.16	1.36	0.432	0.376	0.87	3.20	2.421	0.757	271	270	0.999
Aug	2.31	1.15	0.496	0.526	1.06	3.31	2.718	0.822	244	258	1.058
Sep	2.33	1.16	0.497	0.434	0.87	5.67	5.445	0.961	295	303	1.029
Oct	2.11	1.06	0.500	0.442	0.88	6.00	5.696	0.949	388	425	1.095
Nov	2.16	0.98	0.454	0.302	0.67	6.33	5.264	0.832	320	371	1.158
Dec	1.67	0.65	0.387	0.253	0.65	6.08	5.253	0.864	362	288	0.795
Total	29.27	14.97									
Average	2.44	1.25	0.49	0.42	0.83	5.84	5.50	0.93	318.48	326.32	1.05

Notes: Annual statistics based on total data set, not averages of monthly means.

An event is defined as ending when six dry hours have elapsed.

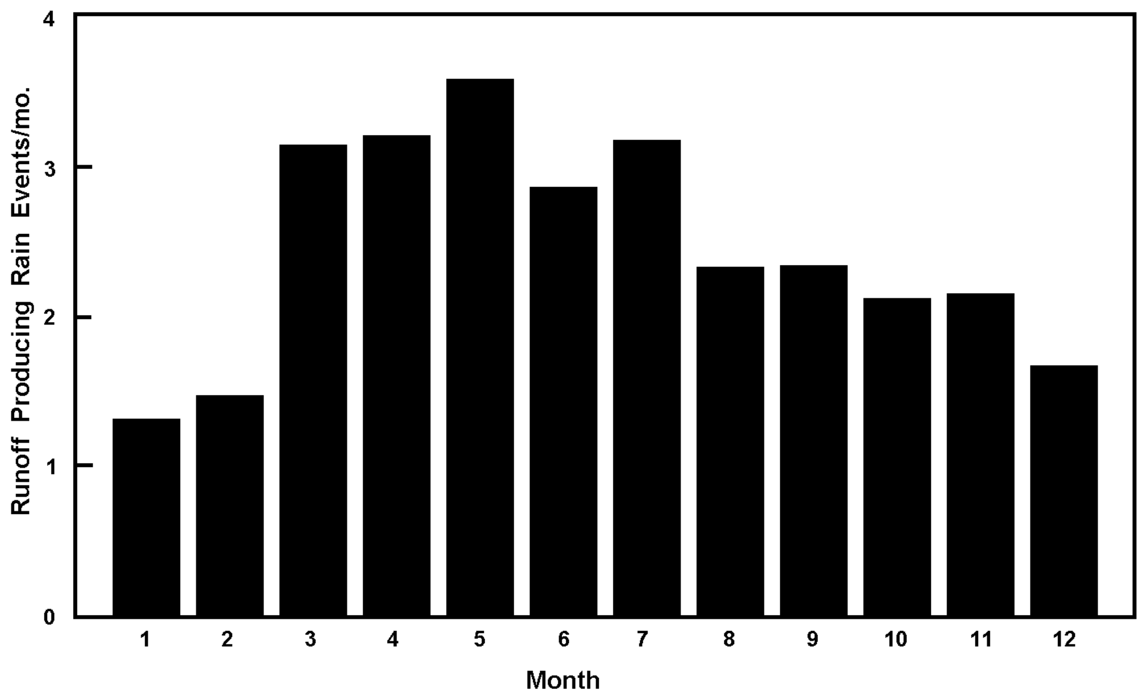


Figure 9-6. Runoff producing events per month in Boulder, CO.

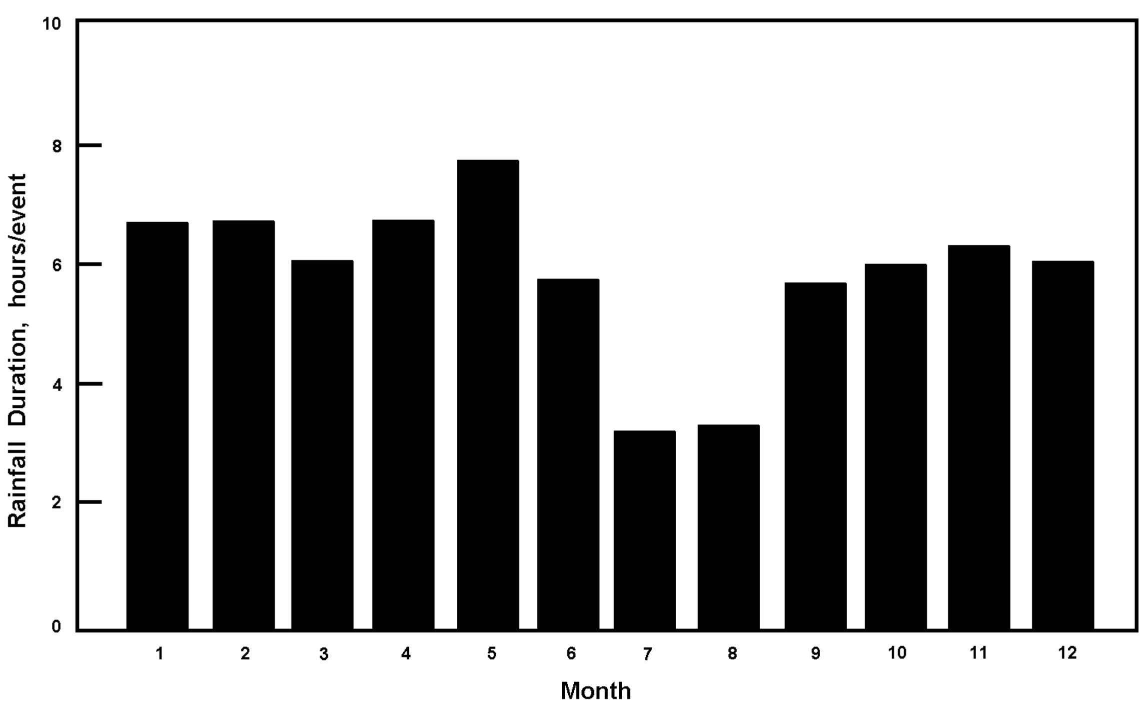


Figure 9-7. Average rainfall duration per event in Boulder, CO.

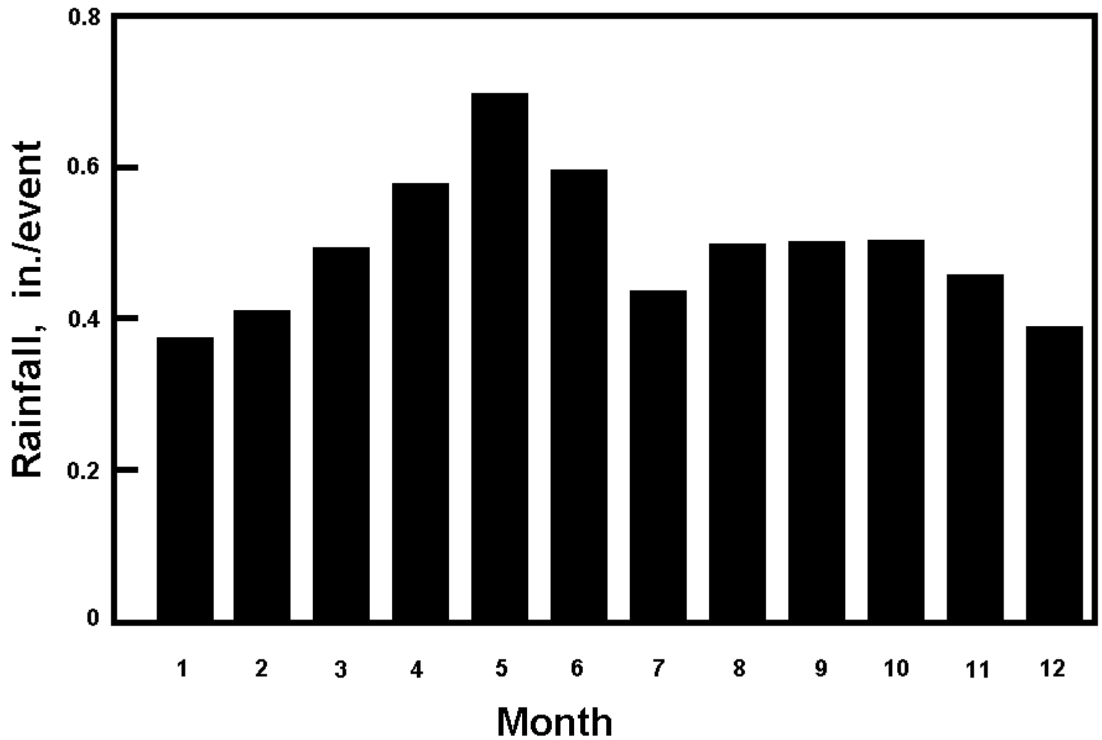


Figure 9-8. Average rainfall per event for Boulder, CO.

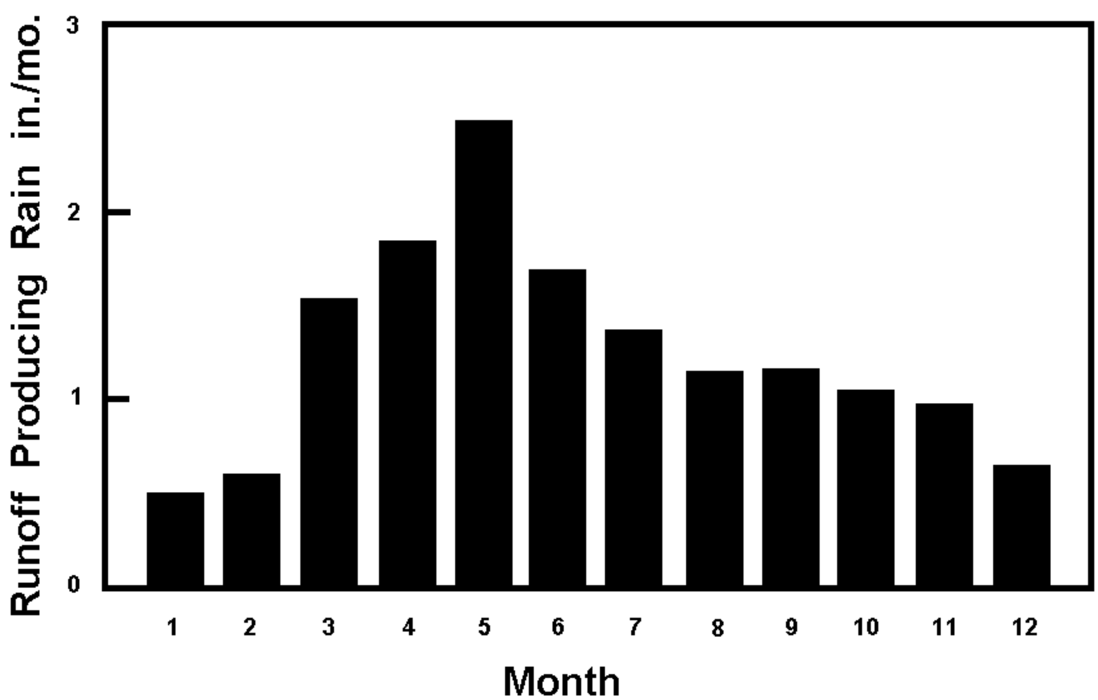


Figure 9-9. Average runoff producing rainfall per month for Boulder, CO.

Table 9-4. Summary of surface water records for Boulder Creek Watershed.

ID	Name	Drainage Area (mi^2)	Period of Record From	Period of Record To	Average Discharge (cfs)	Average Discharge (cfs/mi^2)	Average Discharge (inch/yr)	Upstream Diversion	Storage (ac-ft.)
6726000	N. Boulder C. @ Silver Lake	8.7	1913	1932					
6726500	N. Boulder C. nr. Nederland	30.4	1929	1931					
6725500	Boulder C. at Nederland	36.2	1907	Now	54.3	1.50	20.36	0	Small
6726900	Bummers Gulch nr. El Vado	3.87	1983	Now	0.5	0.13	1.75	0	0
6725500	Boulder C. nr. Orodell	102	1906	Now	86.6	0.85	11.52	Yes, Boulder	11500
6727500	Fourmile C. at Orodell	24.1	1947	1953	6.48	0.27	3.65	?	?
			1982	Now					
6729000	S. Boulder C. nr. Rollinsville	42.7	1910	1918					
			1945	1949					
6729300	S. Boulder C. at Pinecliff	72.7	1979	1980					
6725500	S. Boulder C. nr. Eldorado Spgs.	109	1980		76	0.70	9.46	Big Influence	
6730200	Boulder C. at N. 75th St.	304	1986	Now	90.9	0.30	4.06	Big Influence	Much
6730300	Coal R. nr. Plainview	15.1	1959		4.62	0.31	4.15	None	
6730500	Boulder C. @ Mouth nr. Longmont	439	1927	1949				Big Influence	Much
			1951	1955					
			1978	1990					

Source: Surface water records of the U.S. Geological Survey.
Flows strongly affected by numerous reservoirs and diversions.

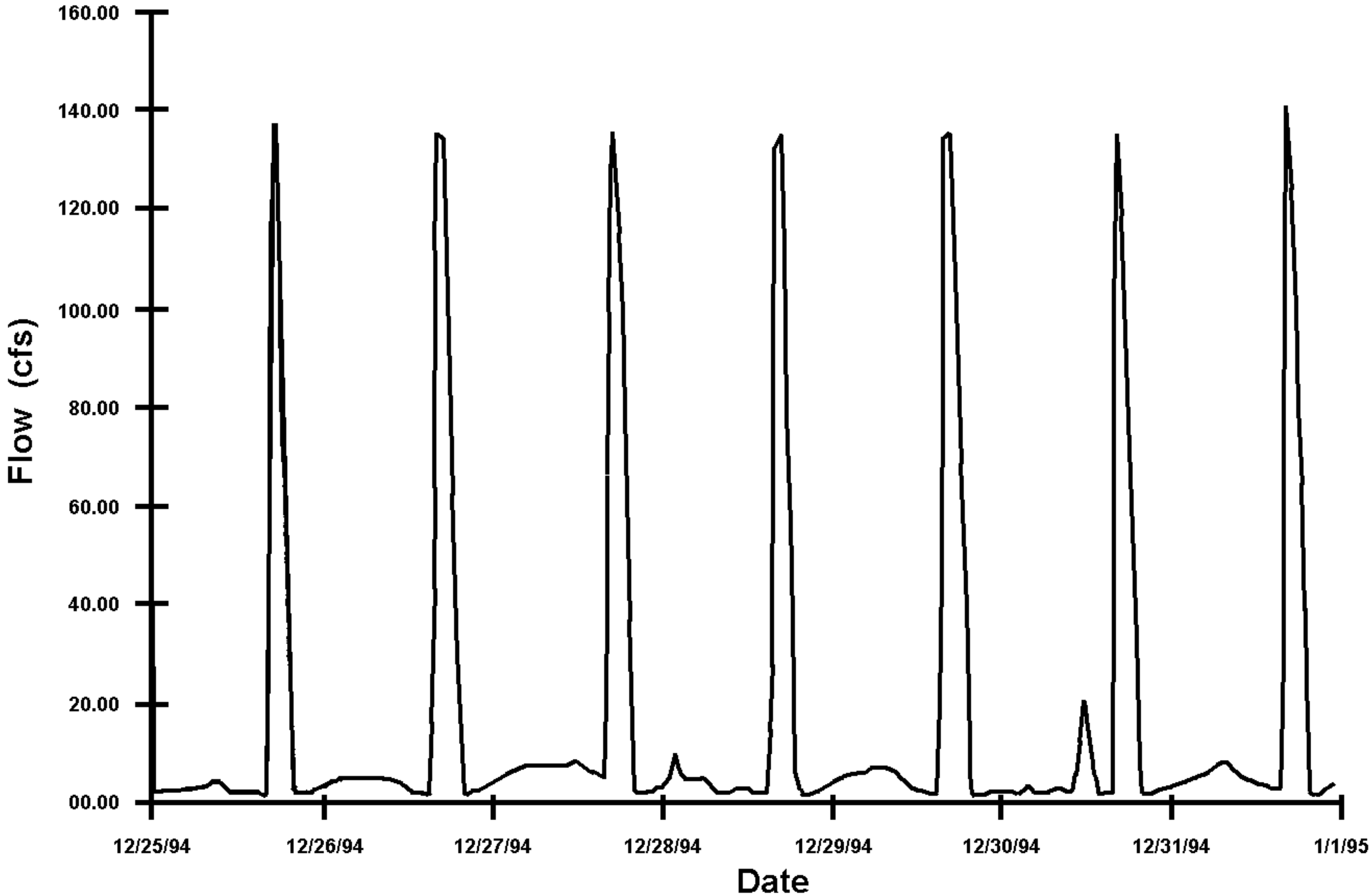


Figure 9-10. Boulder Creek streamflow at Orodell, CO.

The flows in the lower portion of Boulder Creek from 75th St. to its confluence with the St. Vrain River are affected by wastewater treatment plant effluent, Colorado-Big Thompson deliveries from the Boulder Creek Supply Canal, and numerous ditch diversions and return flows. Low flows above 75th St. occur in May and October due to filling of Baseline, Panama, Six-Mile, and Valmont reservoirs. Lowest flows in this section occur in late summer due to diversions for irrigation. Winter flows have increased due to increased releases by PSCO but with a wide range from 1 to 140 cfs over a daily cycle. This pulsed flow occurs only a few hours per day for peaking power.

Middle Boulder Creek has a long-term gage at Nederland just upstream from Barker Dam. This station provides the best estimate of what the unmodified alpine hydrology might look like. Boulder Creek at Orodell includes the contribution of North Boulder Creek. Streamflows at Orodell are affected by the upstream storage in Barker Dam and major diversions for urban water supply and hydropower. Fourmile Creek at Orodell flows can be added to the Boulder Creek at Orodell to get a good estimate of part of the inflow to the urban portion of Boulder Creek.

Within the City of Boulder, numerous diversions take place. Many of the early diversions were for irrigation. These diversions constitute a complex water network, which is difficult to understand as will be discussed in the diversions section.

The Boulder Creek at N. 75th St. gage includes the direct flows in Boulder Creek as the water moves through the City of Boulder. Other components are the sewage effluent from the City, which discharges a few hundred feet above the gage, and numerous other tributary inflows including part of the South Boulder Creek inflow, urban runoff, drainage from local stream channels, and canal inflows to satisfy downstream water rights.

The gage on Boulder Creek at the mouth near Longmont is a discontinued station. Fortunately, there is some overlap with the 75th St. station. Flows in this last section of the stream are heavily affected by agricultural and urban withdrawals and return flows. This section of Boulder Creek between 75th St. gage and Longmont typically loses flow.

South Boulder Creek

The natural runoff of South Boulder Creek at Eldorado Springs is estimated to be about 0.67 cfs/mi^2 (WBLA 1988). The only current station for South Boulder Creek is at Eldorado Springs where South Boulder Creek leaves the mountains. The flows at this station are strongly affected by upstream Gross Reservoir, which is owned by the City of Denver and diverts water from the basin. Downstream of Eldorado Springs, the flow in South Boulder Creek is subject to numerous diversions. These diversions leave South Boulder Creek without water during some months of the year. Because of the lack of stream gages, the quantity diverted and where it enters Boulder Creek is speculative.

Groundwater

To date, relatively little attention has been given to groundwater and the interrelationship between groundwater and surface water. This may change as competition for the available water continues to intensify. No active groundwater monitoring wells are maintained in the study area.

Land Use and Growth Management in Boulder Valley

General

A comprehensive plan has been developed for Boulder Valley (City of Boulder Planning Department and Boulder County Land Use Department 1990). This plan is updated frequently. For planning purposes, the Boulder Valley is divided into the Service Area which is the area serviced by the Boulder Utilities and the Planning Area which includes the Service Area and outlying areas, typically open space areas. The breakdown of land use for the Service Area is shown in Table 9-5 and Figure 9-11. The total service area is 17,225 acres. A roughly equal size of area constitutes the remainder of the total planning area yielding a total planning area of about 35,000 acres.

The City of Boulder has a long tradition of open space land acquisition as chronicled in Figure 9-12 (City of Boulder 1995). In response to rapid population growth during the 1950's and 1960's, Boulder established a "blue line" above which City water would not be provided. The intended effect was to slow the rate of development in the foothills. In 1967, Boulder became the first city in the United States to tax themselves for the acquisition, management, and maintenance of open space land. The increase in the sales tax was 0.4%. In 1989, an additional 0.33% sales tax was approved by the voters for the same purpose. As of 1993, 20,000 acres of land have been protected at a cost of $67 million. By 1995, the total amount of open space land has reached 25,000 acres. The current holdings of the open space program are shown in Figure 9-13.

An ecosystems approach has been used in prioritizing these land acquisitions. With regard to water resources, this has resulted in acquisition of additional water rights which can be used for instream flow needs, reduction in nonpoint loads from lands that would otherwise have been developed, stream restoration, and acquisition of floodplains and wetlands. Recreational use of these open space lands is very high. The 1993 annual level of activity was about 1.7 million visits to this open space land. These recreational uses include hiking, jogging, pet exercising, bicycling, wildlife viewing, horseback riding, and fishing.

In addition to open space acquisition by the City of Boulder, Boulder County has had an aggressive open space acquisition program. This program is supported by sales tax revenues, which currently yield about $4 million per year for open space acquisition. To date, Boulder County has acquired about 35,000 acres of land. Finally, a significant part of the mountain portion of the Boulder Creek Watershed is owned by the U.S. Forest Service. Thus, a very high percentage of the upper watershed land is in public ownership. This provides an excellent opportunity for linked water and land management.

In addition to the open space program, Boulder has an aggressive growth management program. Before growth management, the expected built-out for the water supply system was a population of 250,000. Growth management decisions have reduced this number by 36% to 160,000 (WBLA 1988). This major reduction in growth, coupled with a major open space acquisition program, has greatly reduced the potential impact of urbanization on the water infrastructure system. In the long-run, this is probably the most effective water management tool.

Relative Importance of Urban Land Use

The planning area for Boulder County was divided into 40 drainage basins as shown in Table 9-6. The total drainage area upstream of Boulder is over 84,000 acres (Reaches 1 and 2). Virtually all of this land is undeveloped. Much of it is in public ownership including large U.S. Forest Service holdings. The only current upstream activity is small urban areas, the largest of which is Nederland, a small town located about 20 miles upstream.

The daily runoff was estimated for each of the basins within the City. The western part of the City is grouped into Urban Runoff 1, which consists of eight small drainage areas (Reaches 3-10), the largest of which is 68 acres. Then, Gregory Creek enters Boulder Creek. It drains predominantly undeveloped land, much of it in the protected open space program. The next area draining Boulder Creek is called Urban Runoff 2. It comprises Reaches 12-26 and has a drainage area of 738 acres. Then, Bear Creek enters Boulder Creek. Most of the drainage in Bear Creek is in the open space area. Next, Reaches 28-31 enter Boulder Creek between Bear Creek and Goose Creek. About two thirds of Goose Creek is urban. The last urban runoff group, Urban Runoff 4, enters Boulder Creek between Goose Creek and Wonderland Creek. Then, Wonderland Creek and Fourmile Creek enter Boulder Creek. Lastly, some nonurban lands drain to Boulder Creek between Fourmile Creek and the Wastewater Treatment Plant.

Table 9-5. Land use in the City of Boulder, CO service area – 1995 (City of Boulder Planning GIS Laboratory, unpublished information).

	Area (acres)						
Subcommunity	Residential	Business	Industrial	Open Space	Parks	Public	Total
Central Boulder	2,010	104	0	88	175	154	2,531
North Boulder	1,268	97	63	588	131	55	2,202
U. of Colorado	85	8	0	16	17	508	634
Palo Park	396	23	0	120	10	63	612
Crossroads	252	375	69	30	34	11	771
South Boulder	1,649	33	176	1,280	208	110	3,456
East Boulder	147	5	1,242	207	5	196	1,802
Southeast Boulder	1,862	92	43	218	223	186	2,624
Gunbarrel	1,113	36	1,074	315	36	19	2,593
Total	8,782	773	2,667	2,862	839	1,302	17,225
% of Total	51.0	4.5	15.5	16.6	4.9	7.6	100.0

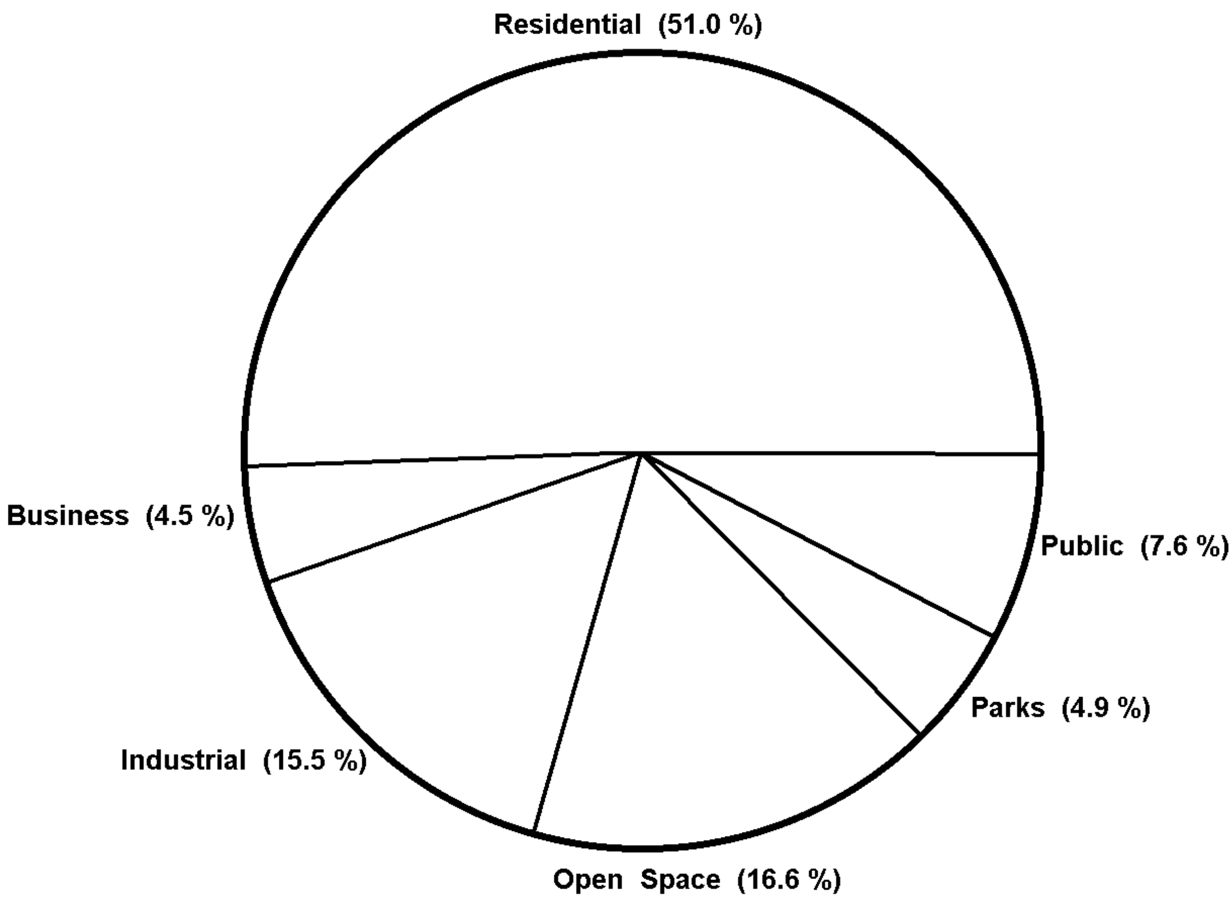


Figure 9-11. Land use in the City of Boulder, CO service area, 1995 (City of Boulder planning GIS laboratory, unpublished information).

1898	Purchase of Chautauqua Park at the foot of Flagstaff Mountain through a bond issue, the beginning of the Boulder Mountain Parks System.
1907	Receipt of 1,600 acres on Flagstaff Mountain from a Congressional grant for the Mountain Parks System.
1910	Frederick L. Olmstead suggests a program for preserving scenic Open Space lands.
1916	Purchase of 1,200 additional acres on Green Mountain and Bear Peak for the Mountain Parks System.
1950-1960	Boulder’s population nearly doubles from 19,999 to 37,718.
1959	Concerned citizens organize to form a group now known as PLAN-Boulder County.
1959	An amendment to the City Charter establishes a “blue line” above which City water will not be supplied. Citizens who helped pass the amendment realized that this would slow development of the foothills, but not stop it.
1960-1970	Boulder’s population again nearly doubles from 37,718 to 68,870.
1963	PLAN Boulder County successfully campaigns for a bond issue to save the 160-acre Enchanted Mesa from development. It is added to the Mountain Parks System.
1965	Citizens defeat a ballot proposal to extend services to a proposed development south of Boulder.
1967	Boulder citizens vote to become the first city in the nation to tax themselves for the acquisition, management, and maintenance of open space land. The measure to permanently increase sales tax by four-tenths of one percent, or \$0.004, passes with 61% of the vote.
1971	An amendment to the City Charter authorizes the City to incur debt to acquire Open Space, allowing for an expanded land acquisition program.
1973	City Council creates the Open Space Board of Trustees to set policies and priorities for acquisition and management of Open Space land.
1978	Boulder Valley Comprehensive Plan (BVCP) states that Open Space shall provide “an important framework for land use planning in the Boulder Valley.”
1986	An amendment to the City Charter provides more permanent protection for Open Space lands, and establishes the Open Space Board of Trustees and the Open Space Department in the Charter, with support of 79% of the voters.
1989	Funding for the accelerated acquisition program passes with 76% of the vote. This adds an additional 0.33 percent sales tax (\$0.0033) for the 15-year period from 1990 through 2004.
1993	Authority to spend all Open Space sales tax revenues and continue to enter into debt for

Figure 9-12. Boulder open space chronology of events (City of Boulder, 1995).

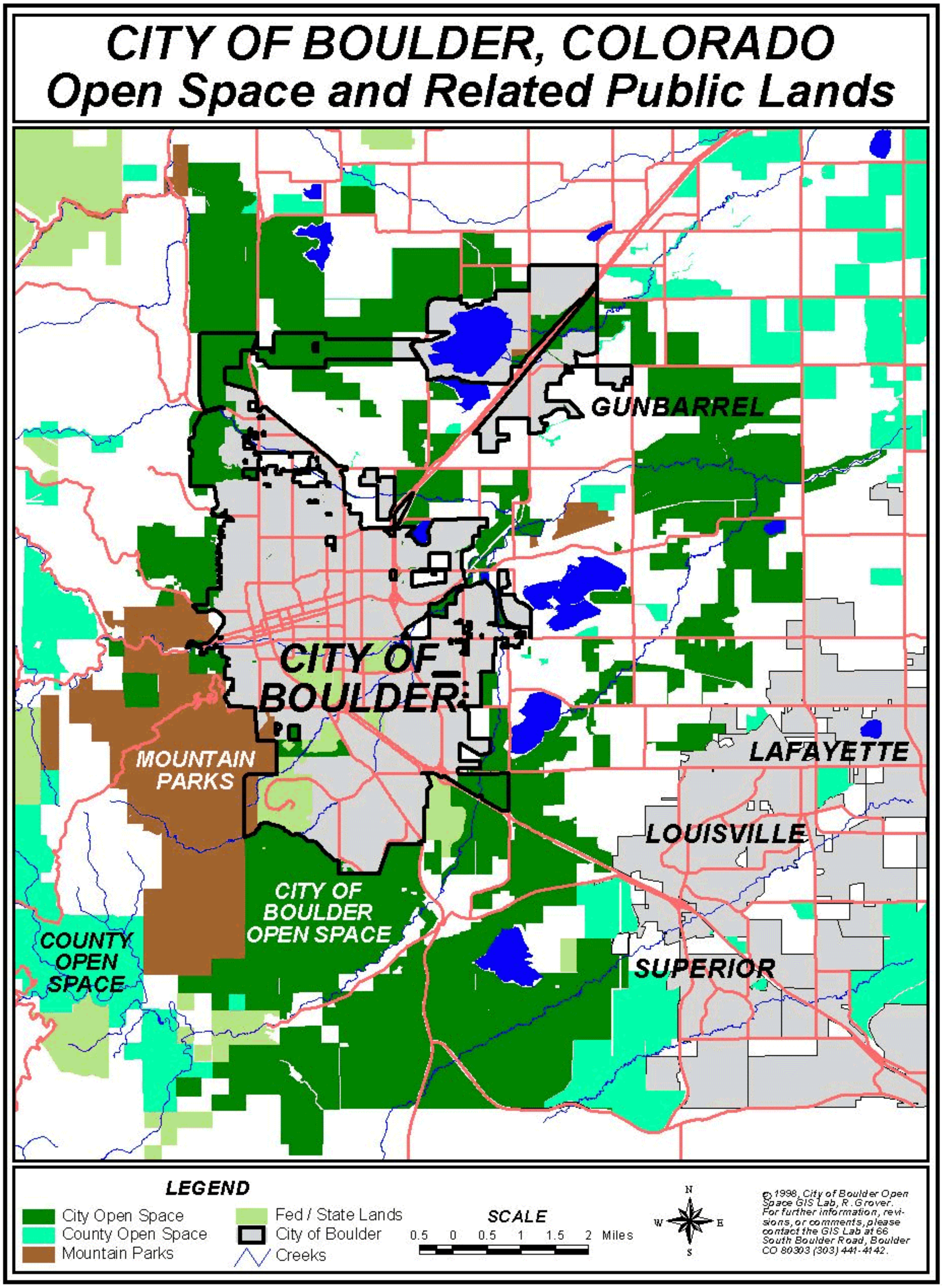


Figure 9-13. Boulder open space and public lands (City of Boulder, 1998).

Table 9-6. Drainage areas for Boulder and Boulder Creek Watershed.

Individual Catchments

Reach	Feet Upstream	Individual Name	Group Name	Area (acres)			Imperviousness (decimal)		
				Total	Urban	Undev.	Average	Urban	Undev.
1	134,200	Boulder C.	Boulder C.	83,200.0	0.0	83,200.0	0.04	0.5	0.04
2	134,100	Sunshine Canyon C.	Sunshine Canyon C.	1,165.0	0.0	1,165.0	0.04	0.5	0.04
3	131,050	DFA 1	Urban Runoff 1	24.9	24.9	0.0	0.50	0.5	0.04
4	130,075	DFA 2	Urban Runoff 1	22.5	22.5	0.0	0.50	0.5	0.04
5	130,020	DFA 19	Urban Runoff 1	9.6	9.6	0.0	0.50	0.5	0.04
6	129,030	DFA 3	Urban Runoff 1	67.2	67.2	0.0	0.50	0.5	0.04
7	129,003	AFA C-5	Urban Runoff 1	67.7	67.7	0.0	0.50	0.5	0.04
8	128,025	AFA C-2	Urban Runoff 1	50.9	50.9	0.0	0.50	0.5	0.04
9	127,095	AFA C-6	Urban Runoff 1	22.3	22.3	0.0	0.50	0.5	0.04
10	127,090	AFA D-1	Urban Runoff 1	66.3	66.3	0.0	0.50	0.5	0.04
11	127,085	Gregory C.	Gregory C.	1,465.6	315.4	1,150.2	0.139	0.5	0.04
12	127,080	AFA C-8	Urban Runoff 2	20.0	20.0	0.0	0.50	0.5	0.04
13	125,010	DFA 4	Urban Runoff 2	35.6	35.6	0.0	0.50	0.5	0.04
14	125,000	C-7	Urban Runoff 2	48.5	48.5	0.0	0.50	0.5	0.04
15	124,015	DFA 5	Urban Runoff 2	42.6	42.6	0.0	0.50	0.5	0.04
16	123,005	D-2	Urban Runoff 2	176.2	176.2	0.0	0.50	0.5	0.04
17	123,000	DFA 6	Urban Runoff 2	41.9	41.9	0.0	0.50	0.5	0.04
18	121,060	D-3	Urban Runoff 2	48.7	48.7	0.0	0.50	0.5	0.04
19	121,058	DFA 7	Urban Runoff 2	19.8	19.8	0.0	0.50	0.5	0.04
20	121,057	C-9	Urban Runoff 2	15.9	15.9	0.0	0.50	0.5	0.04
21	120,004	DFA 8	Urban Runoff 2	23.8	23.8	0.0	0.50	0.5	0.04
22	120,003	C-10	Urban Runoff 2	8.1	8.1	0.0	0.50	0.5	0.04
23	117,025	DFA 9	Urban Runoff 2	45.7	45.7	0.0	0.50	0.5	0.04
24	115,060	C-3	Urban Runoff 2	30.7	30.7	0.0	0.50	0.5	0.04
25	115,045	C-4	Urban Runoff 2	91.2	91.2	0.0	0.50	0.5	0.04
26	115,030	DFA 10	Urban Runoff 2	89.2	89.2	0.0	0.50	0.5	0.04
27	114,000	Bear Creek	Bear Creek	5,273.6	1,456.0	3,817.6	0.167	0.5	0.04
28	113,080	E-1	Urban Runoff 3	99.1	56.0	43.1	0.30	0.3	0.04
29	113,075	DFA 11	Urban Runoff 3	46.1	26.1	20.0	0.30	0.3	0.04
30	113,070	E-2	Urban Runoff 3	166.3	94.0	72.3	0.30	0.3	0.04
31	109,065	DFA 15	Urban Runoff 3	52.9	29.9	23.0	0.30	0.3	0.04
32	108,100	Goose Creek	Goose Creek	3,494.4	2,294.1	1,200.3	0.342	0.5	0.04
33	108,006	DFA 13	Urban Runoff 4	23.8	18.6	5.2	0.40	0.4	0.04
34	108,005	DFA 14	Urban Runoff 4	193.8	151.7	42.1	0.40	0.4	0.04
35	108,000	B	Urban Runoff 4	255.3	199.8	55.5	0.40	0.4	0.04
36	107,099	A	Urban Runoff 4	237.5	185.9	51.6	0.40	0.4	0.04
37	107,095	DFA 18	Urban Runoff 4	18.2	14.2	4.0	0.40	0.4	0.04
38	106,050	Wonderland C.	Wonderland C.	1,222.4	430.5	791.9	0.202	0.5	0.04
39	100,000	Fourmile Canyon C.	Fourmile Canyon C.	6,419.2	781.5	5,637.7	0.096	0.5	0.04
40	91,000	WW Treat. Plt.	WW Treat. Plt.	500.0	65.2	434.8	0.10	0.5	0.04
		Total area above Boulder		84,365.0	0.0	84,365.0			
		Total area in Boulder		20,537.5	7,188.2	13,349.3			
		Total area below Boulder		104,902.5	7,188.2	97,714.3			

Aggregated Areas

Number	Station	Group Name	Acres
1	134,200	Boulder C.	83,200.0
2	134,100	Sunshine Canyon C.	1,165.0
3	128,924	Urban Runoff 1	331.4
4	127,085	Gregory C.	1,465.6
5	120,830	Urban Runoff 2	737.9
6	114,000	Bear Creek	5,273.6
7	112,073	Urban Runoff 3	364.4
8	108,100	Goose Creek	3,494.4
9	107,641	Urban Runoff 4	728.6
10	106,050	Wonderland C.	1,222.4
11	100,000	Fourmile Canyon C.	6,419.2
12	91,000	Wastewater Treatment Plant	500.0
		Total Area	104,902.5

Because of the open space land acquisition program, the public ownership of the upstream drainage area, and the growth management program, Boulder has been able to minimize the amount of urban runoff generation by minimizing urban land use. Only 7,200 acres out of a total of 20,500 acres in the local drainage generate urban runoff. With upstream drainage of over 84,000 acres, only about seven percent of the land use in the Boulder Creek Watershed above 75th St. is urban. Thus, urban runoff would be expected to be a relatively small portion of the total runoff based on land use analysis.

Water Management Infrastructure

Storage

Natural storage in BCW consisted of a few alpine lakes. However, because of the highly variable nature of the streamflow, construction of storage reservoirs was essential. Barker Dam on Middle Boulder Creek was built in 1910. Seven storage reservoirs were built in North Boulder Creek about the same time. Gross Reservoir on South Boulder Creek was built by the City of Denver to store and divert water for its purposes. Within the plains portion of BCW, numerous reservoirs have been built throughout the basin in order to store water including Boulder Reservoir, Valmont Reservoir, and Baseline Reservoir. Boulder Reservoir was built in 1954 at a cost of $1,190,800 as part of Boulder's contribution for participating in the Colorado-Big Thompson Project, which brings water from the north into Boulder Reservoir. Its original capacity was 12,700 acre feet. Overall, there are about 25 to 30 reservoirs in the valley, each one operated to accomplish local or specific objectives within the overall water resources system.

Canals

An extensive canal network has been constructed during the past 140 years. Early canals were built from the mountains to the valleys to maximize gravity flow. Coupled with the storage reservoirs, these canals form a complex water delivery system. Many of the "canals" were parts of the minor tributary system. Thus, the distinction between a "receiving water" and a "canal" is a blurred one at best since these open canals also serve as drainage ditches. This has implications for water quality management.

Control Works

A total of 27 major control works exist in the BCW. Two diversion structures are on North Boulder Creek. These control structures control reservoir releases to the Lakewood pipeline. The main control structure in the upper portion of Middle Boulder Creek is at Barker Dam. This structure directs water into the pipeline, which is shared by the City of Boulder and PSCO. In the valley portion of BCW, diversion structures exist at the mouth of the canyon, at Broadway, and along the downstream portions of the main stem of Boulder Creek. South Boulder Creek has 12 diversion structures on its banks. Each of these diversion structures feeds water into a canal and/or reservoir system which may further branch out to additional canals and associated control structures.

Pipelines

Two major pipelines in the system are located in North Boulder Creek. Lakewood Pipeline was originally installed to protect the City's water supply from contamination by mining activities in the early 1900's. The other pipeline goes from Barker Dam on Middle Boulder Creek to the PSCO generating facilities and the City's Betasso Water Treatment Plant. This 50 cfs pipeline was originally constructed by PSCO which now shares it with the City of Boulder. These two diversions have a major impact on streamflows in the mountain portion of BCW.

Imports and Exports

The major importation of water occurs from the north as part of the Colorado-Big Thompson and Windy Gap Projects. This water enters the Boulder Creek Watershed via an open canal that discharges into Boulder Reservoir north of Boulder. The major export is from Gross Reservoir on South Boulder Creek to the City of Denver. Also, numerous diversions from Boulder Creek occur as the stream enters the city.

Current Water Management System

The current water management system bears little resemblance to the natural system. Reservoirs, canals, diversion structures, and a complex prior appropriation water doctrine have evolved to dictate the operation of the contemporary system.

Water Quantity

Area inhabitants have used BCW for virtually all purposes. Also, BCW has impacted inhabitants through flooding and other undesirable factors. A summary of these activities is presented below.

Municipal Water Supply and Wastewater Return

The City of Boulder began operating a water supply system in 1874. However, even at that early date, much of the water had been preempted for agricultural and mining purposes. Thus, the City's junior water right left them vulnerable during low flow periods. In response, Boulder began to acquire some agricultural water rights and constructed more storage capacity. In response to pollution from upstream mining activities, the City relocated its intake upstream on two occasions. Finally, Boulder placed the intake in the headwaters of the BCW and the water was transported to the City via the Lakewood pipeline, which was completed in 1906. They also acquired the entire headwaters of the watershed to protect the water from pollution.

This system functioned well until the serious drought of the early 1950's forced the City of Boulder to further supplement their system with a water rights exchange agreement, which allowed the City to use more upper basin water in exchange for providing an equivalent amount of water downstream. Also, Boulder acquired significant storage rights in Barker Reservoir from PSCO and the ability to transport this water to their treatment plant via a pipeline. Finally, Boulder joined the Colorado-Big Thompson Project to obtain water from the north. The City built Boulder Reservoir north of Boulder as part of this agreement.

These acquisitions provided Boulder with a major improvement in the reliability of their system. Relatively recent master plans for the water supply system have been prepared by WBLA (1988) and Brown and Caldwell (1990).

The water demand for Boulder for 1992 was 19.73 mgd with peak monthly demand of 32.45 mgd in July as shown in Table 9-7. About 62% of the demand is for indoor use and the remainder is for outdoor use. However, most of the summer water demand is for outdoor use as shown in Table 9-7 and Figure 9-14.

Much of the urban water use is returned to Boulder Creek at 75th St. after treatment. For 1992, the average return flow from the treatment plant was 17.41 mgd. About 5.1 mgd of this total is estimated to be infiltration as shown in Table 9-7 and Figure 9-15. Lastly, the WWTP flow and the streamflow are compared in Table 9-7 and Figure 9-16. The WWTP effluent flow is larger than the streamflow in the colder months of the year.

Agricultural Water Supply

Irrigation using Boulder Creek water is practiced in the valley portion of BCW. Major diversions for agricultural water use occur at eight locations along Boulder Creek as it moves through the City. For 1992, the average diversion for agriculture was 36.64 cfs. These diversions have a major impact on the amount of flow in Boulder Creek because they occur at the western end of the City.

Flood Control

Boulder has been plagued by flooding since its founding because the early settlers located close to Boulder Creek to have easy access for water supply. Smith (1987) has chronicled the evolution of Boulder's flooding problems since its inception. The first recorded flood was in 1864. Subsequent floods in 1867, 1876, and 1885 caused the creek to spread a mile and a half wide. The major flood of record occurred in 1894 with an estimated discharge of 7,400 cfs. This flood did major damage to the town. Continued problems with flooding prompted the City to hire consultants to make recommendations on how best to manage the problem. Mr. Frederick Law Olmstead, Jr. proved to be the most prophetic. In 1910, he recommended a plan, which is very similar to what the City adopted in 1985, 75 years later, that is, a linear park.

Flooding during the second decade of the 20th century broke the City's water line twice. The City remained indecisive for many years in spite of a constant stream of consulting studies, which recommended a wide variety of structural and non-structural solutions. As the City procrastinated, the problem became potentially worse. Nevertheless, progress was eventually made and Boulder has developed a sophisticated stormwater quantity and quality management program.

Table 9-7. Comparison of water use and wastewater flows, 1992.

Month	FLOW IN MGD							
	Water Demand			Wastewater Treatment Plant			Boulder Creek	
	Indoor	Outdoor	Total	Base	Infilt.	Total	Above WWTP	@ 75th St.
Jan	11.74	0.00	11.74	12.31	2.22	14.53	11.80	26.33
Feb	12.31	0.72	13.03	12.31	2.37	14.69	7.40	22.08
Mar	12.31	0.46	12.77	12.31	6.94	19.25	28.01	47.26
Apr	12.31	5.46	17.77	12.31	5.99	18.31	35.72	54.03
May	12.31	13.99	26.30	12.31	5.50	17.81	69.83	87.64
Jun	12.31	13.52	25.83	12.31	6.28	18.59	65.56	84.14
Jul	12.31	20.14	32.45	12.31	6.51	18.82	105.22	124.04
Aug	12.31	13.59	25.90	12.31	7.03	19.34	68.99	88.33
Sep	12.31	15.09	27.40	12.31	6.40	18.71	13.91	32.62
Oct	12.31	6.43	18.74	12.31	4.81	17.12	9.46	26.58
Nov	12.31	0.19	12.50	12.31	3.75	16.06	8.30	24.36
Dec	12.31	0.00	12.31	12.31	3.39	15.70	15.66	31.35
Avg.	12.26	7.47	19.73	12.31	5.10	17.41	36.65	54.07
% of Total	62.2	37.8	100.0	70.7	29.3	100.0		

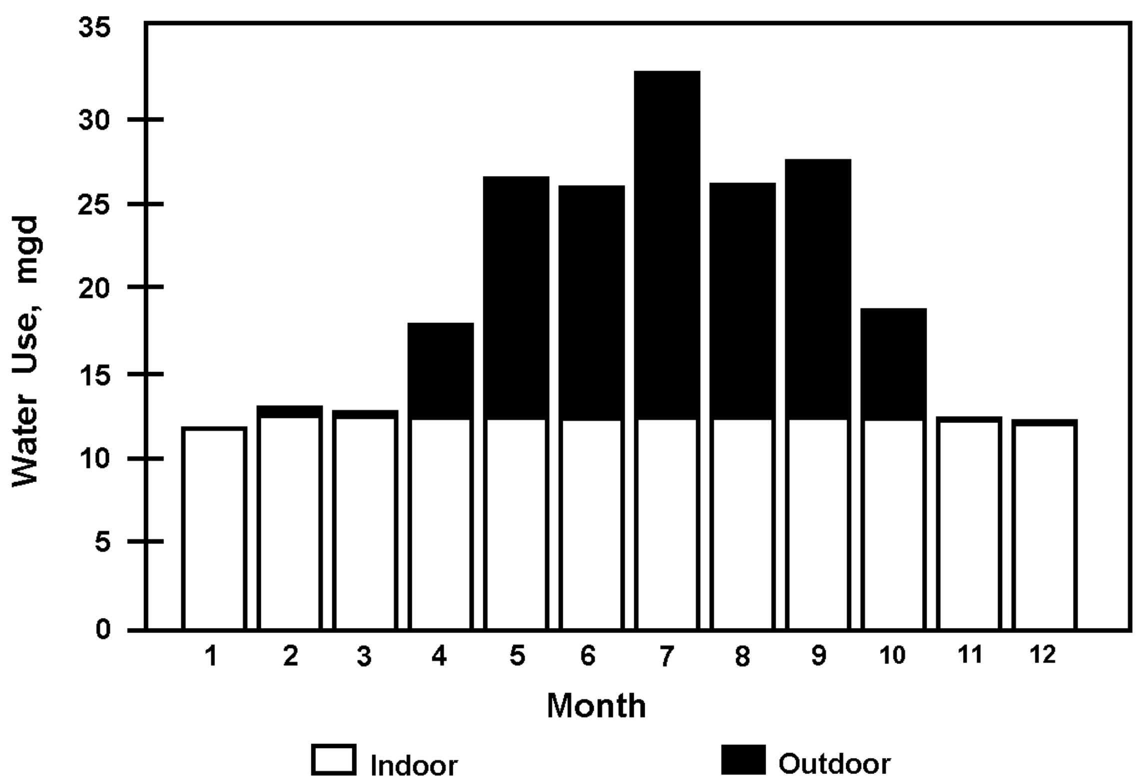


Figure 9-14. Monthly water use for Boulder, CO, 1992.

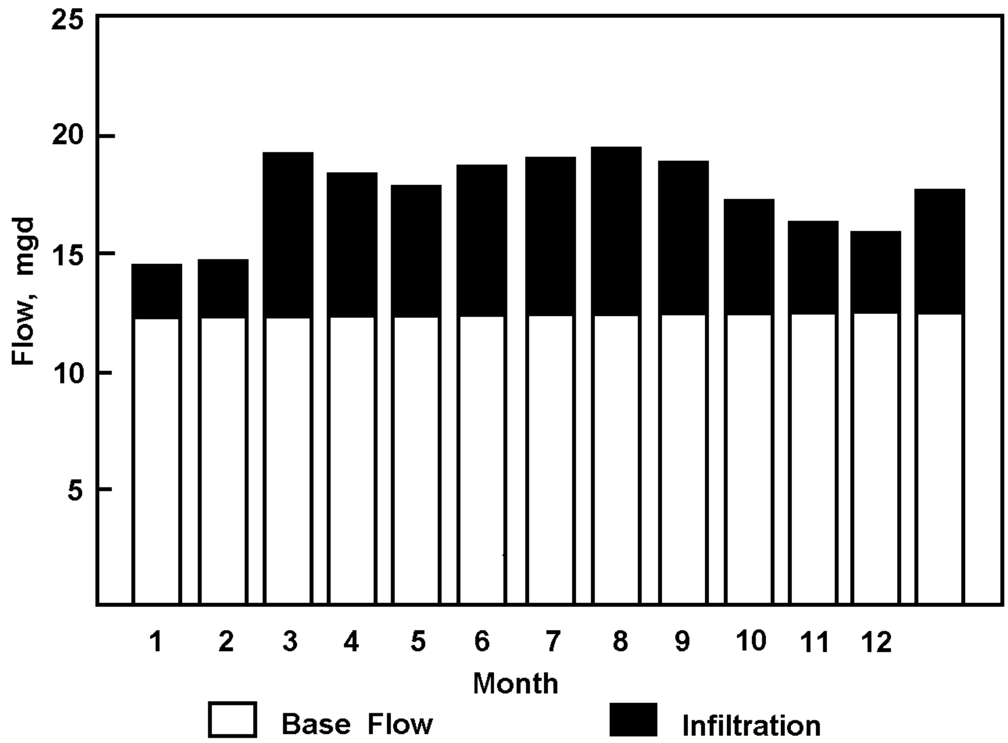


Figure 9-15. Monthly wastewater volumes for Boulder, CO, 1992.

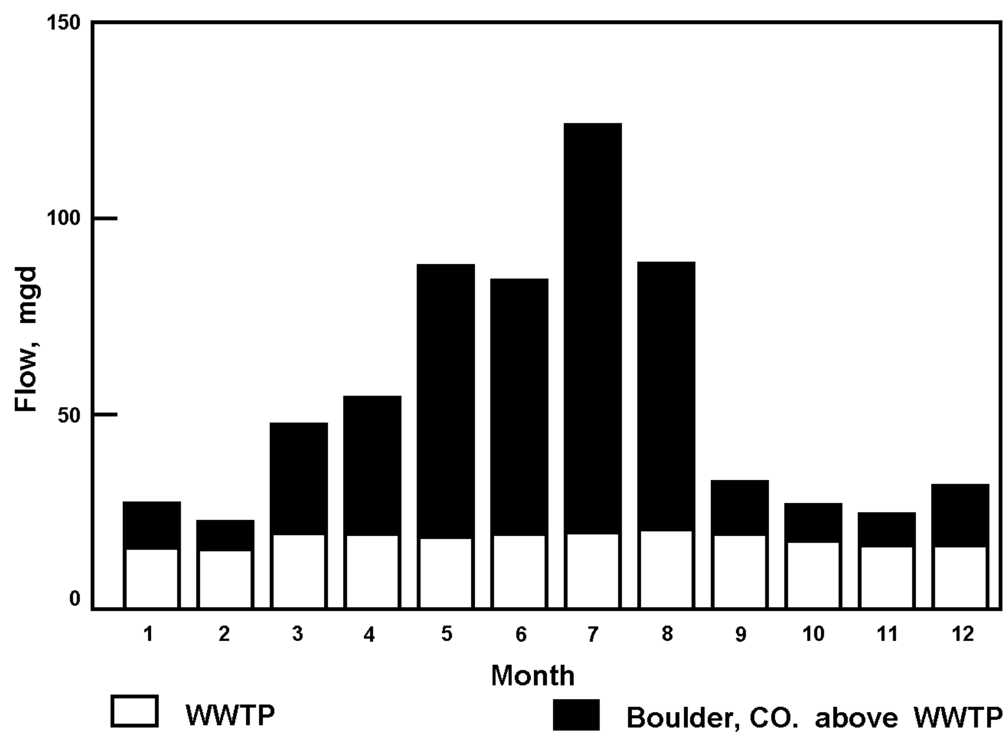


Figure 9-16. Monthly wastewater and Boulder Creek flows, 1992.

However, Boulder remains the most at risk community in Colorado for potential flooding due to its development in relatively high hazard areas and the flashy nature of floods in this area. Boulder has taken a benefit-cost-risk approach to stormwater management. Using a combination of nonstructural and structural controls, they have delineated facilities which can be built and remain in the floodplain. Typically, these buildings are public buildings such as government offices and the library. A floodplain map, shown in Figure 9-17, indicates that much valuable property in downtown Boulder and parts of University housing remain at risk.

Greenway Program

With increased diversions over time, Boulder Creek was literally dried up by mid to late summer. In the 1960's and 1970's, the community began to be concerned about rapid growth. An outcome of that concern was a desire to maintain urban stream corridors as community amenities. Described in this section is the manner in which this desire was articulated in the 1978 Boulder Valley Comprehensive Plan.

An underlying principle was that the functional and aesthetic qualities of drainage courses and waterways shall be preserved and enhanced in a manner compatible with a basically non-structural approach to flood control. In particular, a non-containment approach to flood management was to be followed for Boulder Creek.

Beginning in the 1970's, a succession of plans proposed a trail along the creek. The final design, which emerged in the mid 1980's, called for restoring environmental features and establishing a non-motorized corridor along the creek. A series of objectives were identified including:

1. Create an offstreet non-motorized transportation system.
2. Preserve and enhance fish and wildlife habitat.
3. Protect ecologically sensitive areas.
4. Expand recreational use.
5. Protect water rights of multiple irrigation companies.
6. Maintain and improve flood carrying capacity of the waterway.
7. Protect water quality.
8. Provide opportunities for active and passive recreation.

The final design included strategies to revitalize the creek for fish, wildlife and recreation, including engineering whitewater boating features, enhancing fisheries habitat, and developing paved and gravel pathways to serve bicyclists, walkers, joggers and the disabled. A total of 65 fish habitat improvements were included. Structures included upstream v-dams, angled boulder dams, boulder deflectors, s-dams, and double wing deflectors. Ripple and pool areas provide desirable fish habitat especially during rapid changes in flow due to hydropower generation.

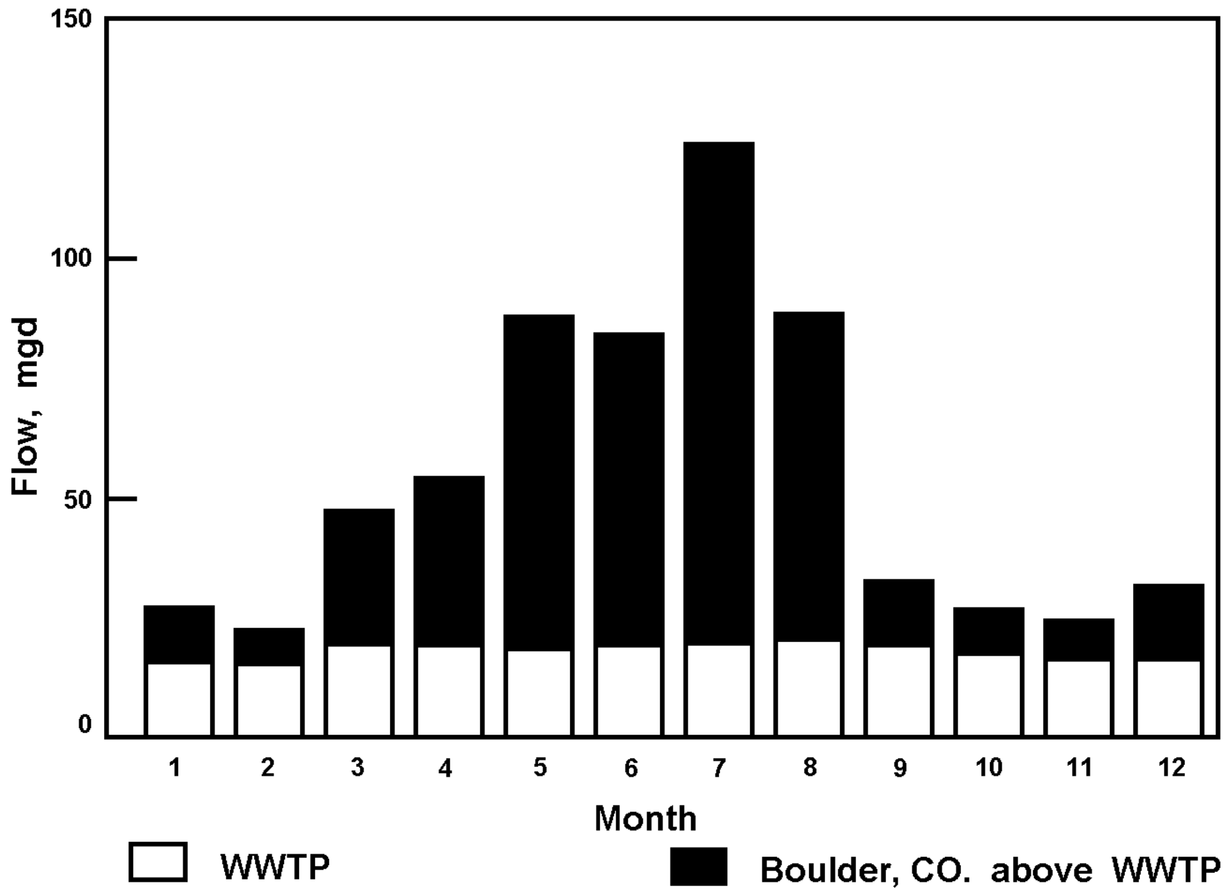


Figure 9-17. Boulder Creek potential flood inundation.

BCW has a very high recreational value to the community, especially after its restoration during the 1980's. A linear park with a bike path were constructed and much instream restoration work was done to help return the stream to a more natural appearance. This work has won a national award for innovative design. Also, Boulder's greenway is one of eight nationally to be featured in a recent book on greenways (Smith and Hellmund, eds. 1993). The Boulder Creek linear park system is heavily used for activities such as walking, jogging, biking and roller blading. Fishing, kayaking and tubing are popular in the upper reaches of Boulder Creek within the City. Boulder Creek was used as the kayak course for the 1995 Olympic Festival.

The original five-mile long Boulder Creek Greenway Project cost $3.3 million with about $1.3 million coming from State Lottery funds. The program continues to grow to include the rest of the Boulder Creek stream system. The current budget is over one million dollars per year. The idea of greenways has spread to other areas. Mayor Webb of Denver has made development of a greenway along the South Platte River as it moves through Denver a cornerstone of his current term in office. The 10 mile long restoration is expected to cost about $50 million and take ten years to complete.

With regard to the required flows for recreational uses, Boulder Creek, from the mouth of the canyon to 55th St., can support the recreational activities listed in Table 9-8.

Table 9-8. Recreational activities supported by flows in Boulder Creek.

Activity	Flow Range (cfs)	Months
Swimming	(1)	(1)
Wading	10-100	June-September
Kayaking	150-300	June-July
Tubing	50-100	July-August
Fishing	15-100	May-September
Fisheries Maintenance	> 15 cfs	May-September
	> 5 cfs	October-April

1) Swimming is not supported because velocities are too high and temperature and depth are too low.

Water quality has not been a major issue. The quality of the water is excellent. Urban runoff quality has not been a major concern. Primary episodes to date deal with spills and deliberate discharges of hazardous materials, such as paint, into the storm drains. In contrast, maintaining minimum instream flows has been a high priority concern. Prior to a major instream restoration effort in the mid 1980's, base flow in Boulder Creek as it moved through Boulder was often zero. Thus, an obvious part of stream restoration was to have adequate base flows, especially in late summer.

Hydropower

Hydropower is an important component of the BCW water resource system. PSCO provides most of the energy for the Boulder Valley and owns and operates Barker Dam on Middle Boulder Creek. Water is released from Barker Dam to a pipeline, which is used to transport the water to the generating facilities. The water is returned to Middle Boulder Creek just upstream from the Orodell gage. PSCO also diverts water from South Boulder Creek and Boulder Creek at 28th St. to Valmont Reservoir which is used for cooling water for its electric generating facilities in Boulder. PSCO has agreements with the City of Boulder for joint utilization of the storage in Barker Dam and for the pipeline to the generating facilities.

Hydropower releases can cause major variability in flows in Boulder Creek. During the winter months, flows are released only part of the day to meet early evening peaking requirements. These flows are pulsed to permit efficient use of the turbines. The hourly flows for Middle Boulder Creek at Orodell for late December 1994 are shown in Figure 9-10. The daily flows range from near 0 cfs for most of the day to about 140 cfs for the early evening hours. The flows for December 25, 1994 are shown in Figure 9-18. From midnight to about 5 pm, the flow in Boulder Creek is a few cfs. From 5 pm to 9 pm, the flow increases rapidly to about 136 cfs and then decreases rapidly back to 0 at about 9 pm. This highly variable flow would be expected to have a significant impact on the fisheries (WBLA 1988). Another concern is the diversion of Boulder Creek water at 28th St. to replenish Valmont Reservoir during the non-irrigation season. This diversion reduces low flows in the stream during fall and spring. Early fall, in particular, is a sensitive period for the receiving water.

Instream Flow Needs

As development in BCW proceeded, more of the available water resource was appropriated for the beneficial uses described above. These other uses left significant sections of BCW with little or no water during parts of the year. The cumulative impact of these diversions is that major problems occur with respect to fish and macroinvertebrate survival in all but the peak flow months from May through July as follows (Rozaklis 1994):

1. North Boulder Creek: Zero flow past Lakewood from October-March.

2. Middle Boulder Creek: Zero flow below Barker Dam from October-April.

3. Main Boulder Creek: Inadequate flow through the City. Periods of low or zero flow in late summer.

4. South Boulder Creek: Zero flow below Eldorado from November-March. Also, zero flows during latter part of the summer.

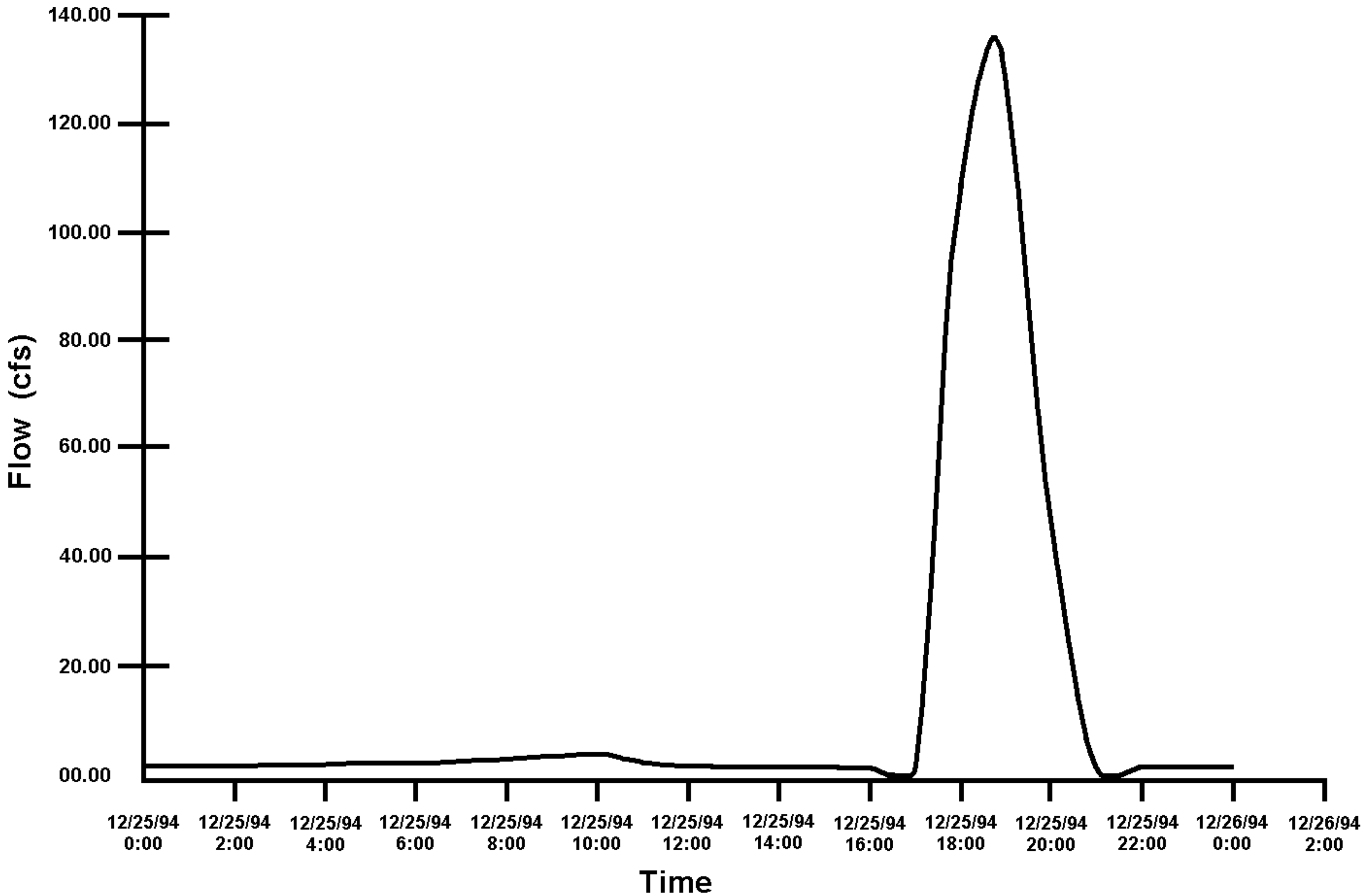


Figure 9-18. Flow in Boulder Creek at the Orodell gauging station, December 25, 1994.

Recognition of this problem and the concomitant desire to restore Boulder Creek led the City to embark on an aggressive program to increase low flows in BCW. After five years of negotiations, the City was able to transfer its water rights to provide a minimum flow of 15 cfs in Middle Boulder Creek and minimum flows in other parts of the BCW system. This water will be available for instream flow needs in all but the most serious droughts. If such a drought occurs, the City can use this water for essential water needs. The present value of these water rights transfers is about $14 million, a significant investment for the City of Boulder.

Understandably, restoring base flows for instream needs is the top priority for a stream restoration program. This water is of excellent quality. The next steps include:

1. Improved monitoring to verify that these instream flows are being maintained.
2. Improved accounting methods for tracking water movement through BCW.
3. Reducing extreme flow variability from pulsed hydropower releases.
4. Obtaining more capacity in Barker Reservoir to better manage instream flow needs in Middle and South Boulder Creeks.
5. Increased attention to water quality management along with water quantity and land management. Nonpoint pollution appears to be the most pressing concern.

Stream restoration is a vital part of the instream flow augmentation program. The required flows to support various instream activities depends upon the nature of the stream. If the stream has been channelized into a trapezoidal cross section, then it is not as desirable from a fishing or boating point of view. With a restored stream system with ripples and pools, the minimum required flow is about three to five cfs whereas it is about 15-20 cfs without stream restoration. Similarly, the kayaking course with restoration requires significantly less flow (20-30 cfs) instead of more than 100 cfs without restoration (Lacy 1995).

Importation of Water
Boulder Creek receives imported water from the Colorado-Big Thompson Project. This water is delivered to Boulder Reservoir north of Boulder. Some of this water is used by the City of Boulder with the balance directed to other users. The Boulder Supply Canal transfers water from Boulder Reservoir to Boulder Creek just upstream of the Wastewater Treatment plant. This water provides a major increase in the streamflow during the warmer months of the year.

Overall Water Budget for Boulder
In order to understand integrated watershed management, a fairly complete water budget for the urban area is essential (McPherson 1973). Calendar year 1992 was chosen because of the availability of data. It was a drier than average year. The key sources and sinks of the water budget are discussed first followed by presentation of annual, monthly, daily, and hourly water budgets.

Sources

1. Boulder Creek at Orodell: This input is measured by a USGS gage. The Orodell station is downstream of North Boulder Creek and therefore includes this source. The natural flow at Orodell has been significantly altered by upstream diversions for municipal water use.

2. Fourmile Creek: This input is measured by a USGS gage.

3. South Boulder Creek: This input is not measured. It is assumed to be zero. Except in wetter years, the entire flow in South Boulder Creek is utilized for needs of area inhabitants.

4. Urban Runoff: This input is estimated based on a very rough estimate of contributing land use. The estimate will be updated with better data.

5. Wastewater Treatment Plant: This input is measured. A significant part of the wastewater flow is infiltration and inflow.

6. Boulder Reservoir: Deliveries to Boulder Creek to satisfy downstream water users. This inflow enters Boulder Creek just upstream of the wastewater treatment plant near 75th St.

Sinks

1. Diversions: These diversions occur at Canyon Mouth, Broadway, and along Boulder Creek between Broadway and 75th St. These data are obtained from the State Engineer's office.

2. Boulder Creek at 75th St.: These are measured flows at a USGS gage.

Annual Water Budget

The annual water budget for calendar year 1992 is shown in Table 9-9 and Figure 9-19. The total estimated sources entering Boulder Creek above 75th St. are the upstream flow of 54.55 cfs, the wastewater treatment plant return flow of 26.98 cfs, the Boulder Supply Canal imported water from the CBT project of 29.29 cfs, and the estimated stormwater runoff of 7.2 cfs. Urban runoff is estimated to be 6.17 cfs out of the total of 7.2 cfs of local runoff. A simple rainfall-runoff relationship was used to estimate the runoff. This simple method was used since the data on land use and imperviousness are only approximate. Also, no direct rainfall-runoff measurements are available.

The sinks of water are the diversions from Boulder Creek. The total of diversions for calendar year 1992 was 36.64 cfs averaged over the entire year. Most of these diversions occur during the irrigation season. This water budget ignores groundwater influences since no data are available. Also, the inflow from South Boulder Creek is estimated to be zero for 1992, a relatively dry year.

Of all of the above items, only the runoff is estimated. All of the other items in the water budget are measured. The overall result of the annual water budget is an estimated total sources of 118.0 cfs and total outflows of 120.9 cfs, leaving unaccounted for a total of 2.9 cfs of inflow. This inflow is some combination of stormwater runoff and groundwater inflow. Lacking better measurements, the nature of this residual is unknown.

The error in the annual water budget is less than 3%. Thus, some statements can be made about the expected relative importance of urban runoff. Urban runoff averages about six cfs over the entire year. By comparison, the WWTP effluent is 26.98 cfs, over four times larger. Of course, urban runoff occurs infrequently (about 2% of the time). Thus, it takes on greater relative importance when it does occur.

Monthly Water Budget

The monthly water budget for CY 1992 is shown in Table 9-10 and is plotted on Figure 9-20. The errors are random. The predictions follow the measured outflow fairly closely. The monthly budgets reflect flow in Boulder Creek at 75th St., the downstream boundary of the City. The flows within the City are significantly less since the Boulder Supply Canal and the WWTP provide major inputs of water. The estimated monthly flow within the city (at 28th St.) is shown in Table 9-11 and the associated time series is shown in Figure 9-21. Much of the inflow to the city is diverted above 28th St. however, most of the urban runoff enters Boulder Creek downstream of the city. Thus, the relative importance of urban runoff is still small as shown in Table 9-10.   Prevailing average monthly flows at 28th St. during the late summer and early fall are in the 10 to 20 cfs range.

Table 9-9. Overall water budget for calendar year 1992 (flow in cfs).

Sources, Average Flow Rate				
		Urban	Other	
1	Sunshine	0.00	0.08	
2	Urban Runoff 1	0.29		
3	Gregory	0.28	0.08	
4	Urban Runoff 2	0.65		
5	Bear Creek	1.29	0.27	
6	Urban Runoff 3	0.11	0.01	
7	Goose Creek	2.03	0.08	
8	Urban Runoff 4	0.40	0.01	
9	Wonderland C.	0.38	0.06	
10	Fourmile C.	0.69	0.40	
11	WWTP	0.06	0.03	
Total Urban & Other Runoff		6.17	1.03	
Total Runoff			7.20	
Upstream			54.55	
WTP off			26.98	
Bo. Sp. Canal			29.29	
Total Source				118.02
Sinks, Average Flow Rate				
1	Anderson	501	3.97	
2	Boulder Lefthand	513	2.43	
3	Boulder White Rock	516	9.59	
4	Farmers	525	7.48	
5	Green	528	2.71	
6	Silverlake	603	1.23	
7	Butte Mill	518	2.11	
8	N. Boulder Farm	543	7.10	
Total Sinks				36.64
Computed Flow (sources-sinks)				81.38
Observed Flow @ 75th gage				84.24
Residual (observed-computed)				2.86

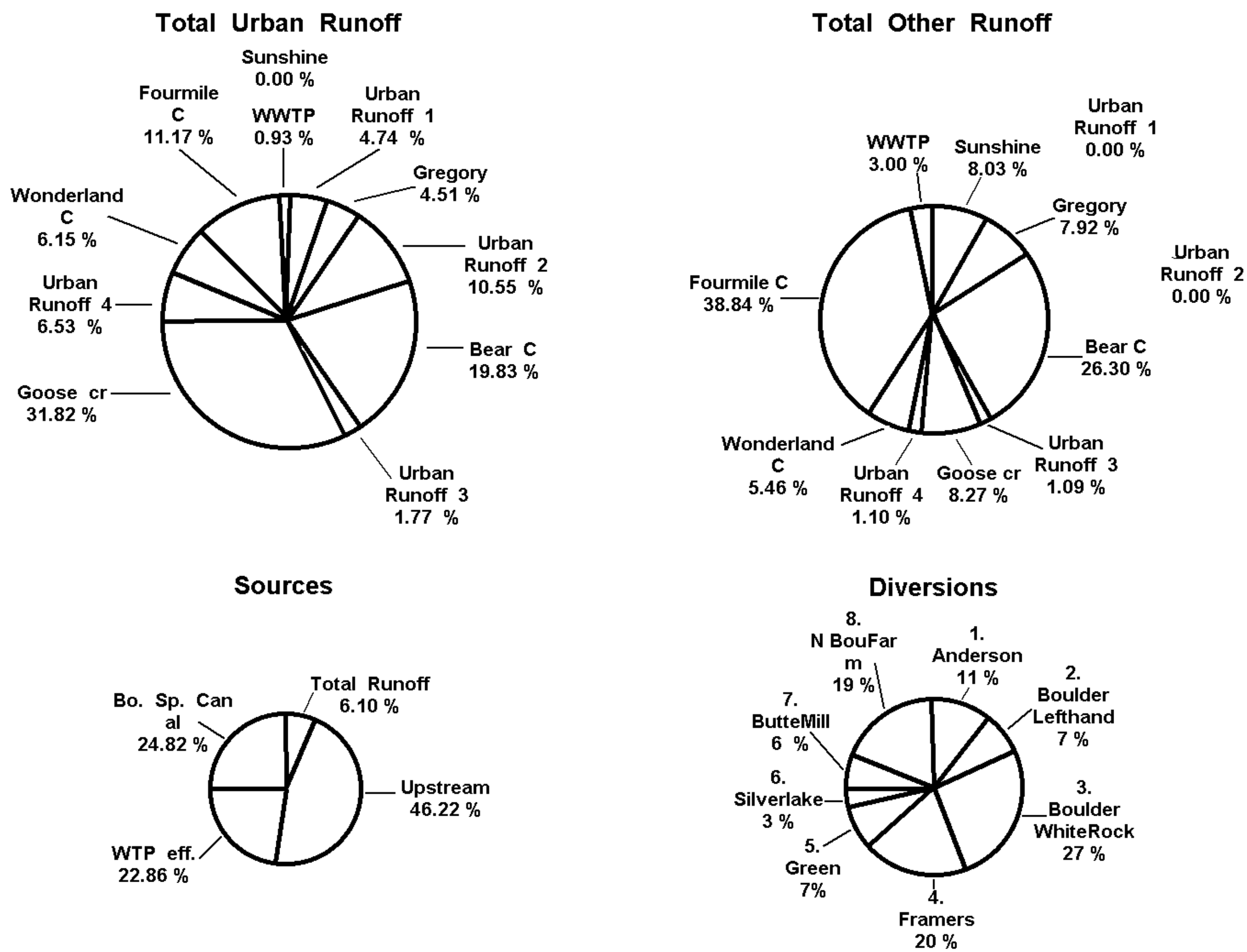


Figure 9-19. Overall water budget for calendar year 1992.

Table 9-10. Measured and computed monthly flowrates in 1992.

Month	Average Flow (cfs)			Residual as % Of Computed
	Computed	Observed	Residual	
Jan	38.45	40.74	-2.30	-6
Feb	38.94	34.28	4.67	12
Mar	72.74	71.71	1.03	1
Apr	62.79	83.47	-20.68	-33
May	125.03	135.35	-10.33	-8
Jun	101.63	127.13	-25.51	-25
Jul	182.38	192.35	-9.97	-5
Aug	149.47	140.52	8.96	6
Sep	43.79	50.77	-6.97	-16
Oct	49.75	41.13	8.62	17
Nov	54.35	37.70	16.65	31
Dec	52.69	48.52	4.17	8

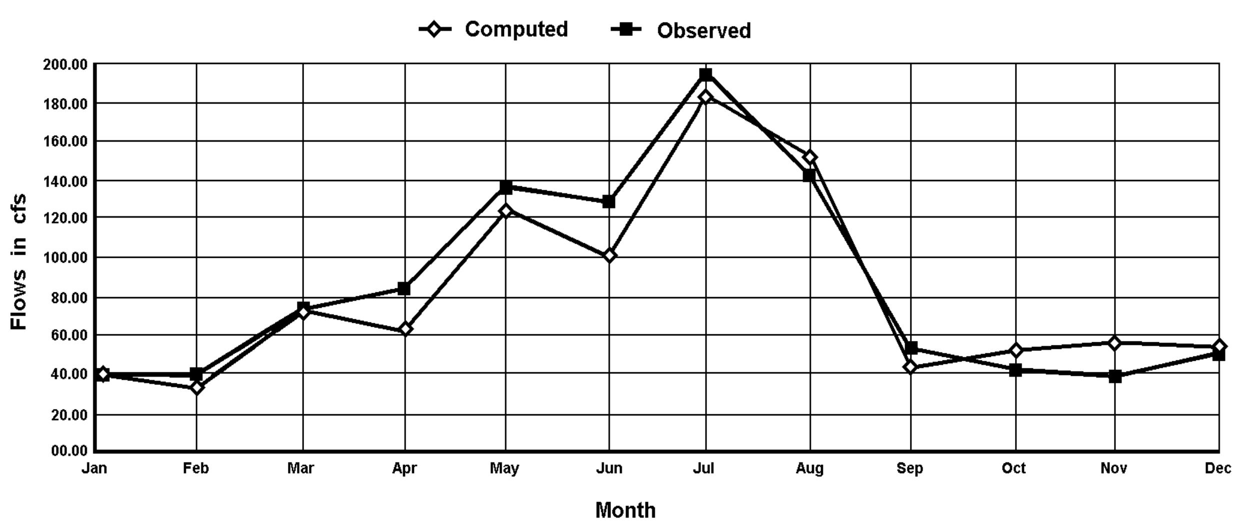


Figure 9-20. Boulder Creek monthly flows in 1992.

Table 9-11. Monthly flows in Boulder Creek at 28th St. for calendar year 1992.

	Month											
	Jan-92	Feb-92	Mar-92	Apr-92	May-92	Jun-92	Jul-92	Aug-92	Sep-92	Oct-92	Nov-92	Dec-92
Sources (cfs)												
Sunshine	0.03	0.00	0.30	0.02	0.10	0.04	0.05	0.17	0.00	0.04	0.17	0.05
Urban Runoff 1	0.09	0.00	1.08	0.07	0.37	0.14	0.19	0.62	0.00	0.15	0.59	0.18
Gregory Urban	0.09	0.00	1.03	0.07	0.35	0.13	0.18	0.59	0.00	0.14	0.56	0.17
Other	0.03	0.00	0.30	0.02	0.10	0.04	0.05	0.17	0.00	0.04	0.16	0.05
Urban (Runoff 1 & Gregory)	0.18	0.00	2.11	0.14	0.72	0.28	0.37	1.21	0.00	0.29	1.15	0.34
Urban Runoff & Other	0.23	0.00	2.72	0.18	0.92	0.36	0.48	1.56	0.00	0.38	1.47	0.44
Upstream	13.67	16.20	21.96	54.57	137.90	133.30	109.77	69.13	33.63	23.45	15.03	24.06
Total Sources	13.90	16.20	24.68	54.75	138.83	133.66	110.25	70.69	33.63	23.83	16.50	24.50
Sinks (cfs)												
501 Anderson	0.00	0.00	5.69	11.76	6.58	6.54	5.87	4.02	3.66	3.23	0.00	0.00
513 Boulder Lefthand	0.00	0.00	0.00	1.94	8.61	11.47	1.95	3.23	1.70	0.33	0.00	0.00
516 Boulder Wrock	0.00	0.00	0.00	7.85	37.22	45.03	22.92	1.97	0.00	0.00	0.00	0.00
525 Farmers	0.00	0.00	0.00	0.00	11.68	26.63	29.51	16.51	4.99	0.00	0.00	0.00
526 Green	0.00	0.00	0.00	0.00	5.87	6.92	5.99	8.18	3.78	1.65	0.00	0.00
603 Silverlake	0.00	0.00	0.00	0.00	1.80	3.58	3.90	3.02	2.47	0.00	0.00	0.00
543 N. BouFarm	0.00	0.00	0.00	0.70	16.19	19.58	23.82	15.70	7.57	1.22	0.00	0.00
Total Sinks	0.00	0.00	5.69	22.25	87.96	119.75	93.96	52.63	24.17	6.43	0.00	0.00
Flow at 28th Street	13.90	16.20	16.27	32.50	50.87	13.91	16.29	18.06	9.46	17.40	16.50	24.50

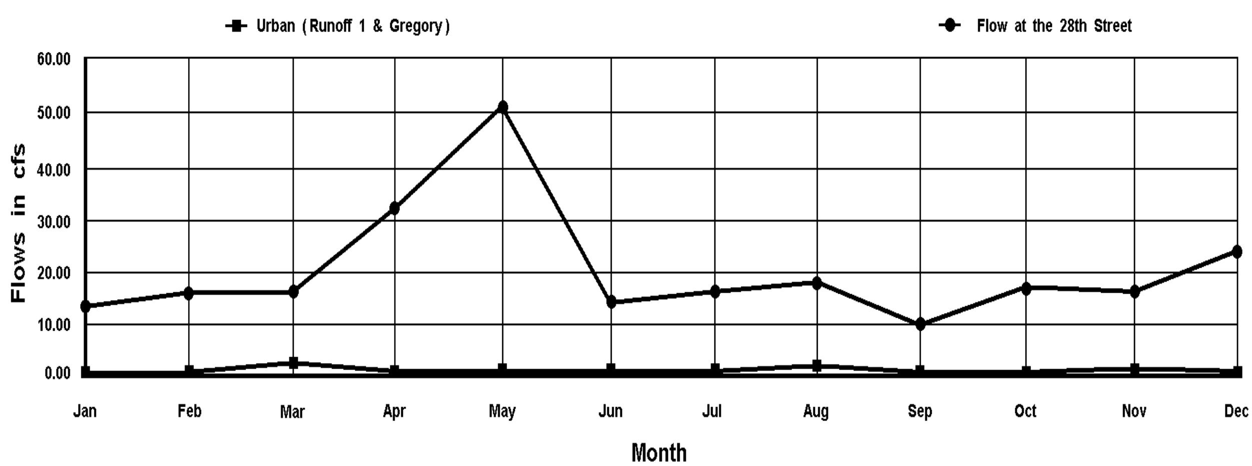


Figure 9-21. Monthly flows in Boulder Creek at 28th St. for calendar year 1992.

The results of the monthly water budget show the dramatic influence of human activities on the flows in Boulder Creek. The 1992 monthly flows above the City of Boulder, within the City of Boulder, and downstream of the City of Boulder are shown in Table 9-12 and Figure 9-22. The streamflows differ dramatically. The inflow above the city is diverted before the stream moves through much of the city. The flow downstream of the city is over four times larger due to water import and the wastewater return flow. Thus, three distinctly different hydrologic environments exist even though the total distance from above to below the city is only about eight miles. The upper and within the city stations are only two miles apart. The magnitude of the human-induced sources within the Boulder study area are shown in Table 9-13 and Figure 9-23. The wastewater treatment plant return flow is relatively constant at 26.97 cfs. However, the diversions and imports vary widely with virtually all of these flows occurring during the irrigation season. On an annual average, the diversions are the largest component followed by the imports. Recall that the estimated urban runoff is about six cfs, far less than these values.

Daily Water Budget

Lastly, a daily water budget was done for calendar year 1992. The results are summarized here. The predicted versus measured flows track fairly well. Notable differences occur during storm periods, especially in the colder months, when the precipitation is actually snow with entirely different runoff patterns. Critical water quality conditions occur during the late summer and early fall so attention was focused on these months. The August results indicate that the maximum actual daily flow at 75th St. was 250 cfs, one half of the predicted maximum flow of 500 cfs. This peak was in response to the largest single rain event of the year. Typical flows decreased from about 200 to 50 cfs over the month. The dominant terms in the water budget for August are the import and export of water for irrigation. Urban runoff is still a relatively small amount. Boulder Creek flows continued to decrease in September to about 40 cfs. During October, the main source of flow in the stream is the WWTP return flow. The Boulder Supply Canal deliveries declined as the irrigation season began to end.

Hourly Water Budget

Only a few cfs of flow are available in Boulder Creek as it passes through the city in late summer and early fall. However, it is important to understand the water budget, not only on a daily basis, but also to do an hourly accounting. From October to March, PSCO releases water to Boulder Creek in pulses for hydropower peaking purposes during the early evening hours. Thus, while the average daily inflow might be 10 to 15 cfs, the actual flow pattern is 140 cfs for two to three hours and zero flow the rest of the day as shown in Figure 9-18. Thus, the fish in Boulder Creek must adapt to very wide swings in flow even on an hourly basis. Similar conditions would occur in other streams where hydropower is generated. Such extreme daily flow swings would tend to have a more significant impact on the fish than urban runoff because of their much greater frequency.

Table 9-12. Monthly flows in Boulder Creek for calendar year 1992, above, within and below the City of Boulder (in cfs).

Month	(1)	(2)	(3)
	Mean Monthly Flows in Boulder Creek, 1992		
	Above Boulder	Within Boulder	Below Boulder
Jan-92	13.67	13.90	40.74
Feb-92	16.20	16.20	34.28
Mar-92	24.68	16.27	71.71
Apr-92	54.75	32.50	83.47
May-92	138.83	50.87	135.35
Jun-92	133.66	13.91	127.13
Jul-92	110.25	16.29	192.35
Aug-92	70.69	18.06	140.52
Sep-92	33.63	9.46	50.77
Oct-92	23.83	17.40	41.13
Nov-92	16.50	16.50	37.70
Dec-92	24.50	24.50	48.52
Average	55.10	20.49	83.64

1. Measured flow above Boulder. Stream mile = 25.5.
2. Estimated flow at 28th St. Stream mile = 23.5.
3. Measured flow below Boulder at 75th St. Stream mile = 17.5.

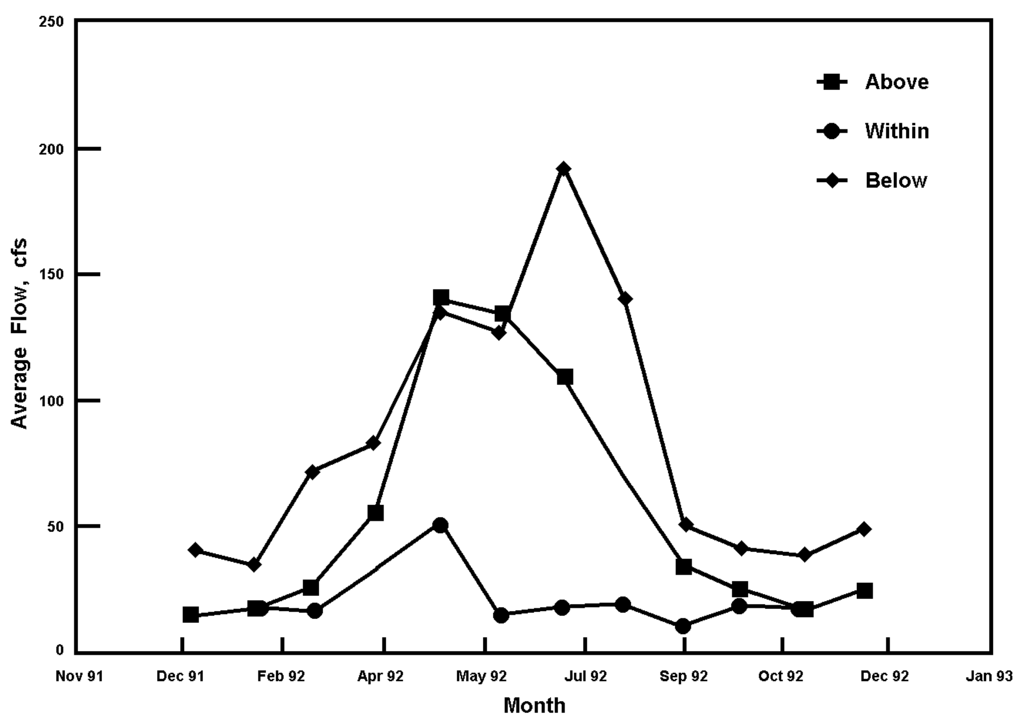


Figure 9-22. Monthly flows in Boulder Creek for calendar year 1992, above, within, and below the City of Boulder.

Table 9-13. Total sources of flow, Boulder Creek, CO, 1992 (in cfs).

Month	Local		Total Runoff	Upstream Inflow	WWTPeff	BsupCanal	Total Sources	Runoff (%)	Runoff Producing Rainfall (inches)
	Urban Runoff	Other Runoff							
Jan	1.94	0.32	2.26	13.67	22.51	0.00	40.71	4.77	0.41
Feb	0.00	0.00	0.00	16.20	22.74	0.00	38.94	0.00	0.00
Mar	22.83	3.79	26.62	21.96	29.81	0.03	105.05	21.73	4.82
Apr	1.52	0.25	1.77	54.57	28.35	0.35	86.80	1.75	0.31
May	7.77	1.29	9.08	137.90	27.58	45.40	229.01	3.39	1.84
Jun	2.99	0.50	3.48	133.30	28.79	62.47	231.52	1.29	0.61
Jul	4.03	0.67	4.70	109.77	29.16	137.97	286.29	1.41	0.85
Aug	13.12	2.18	15.30	69.13	29.96	91.65	221.34	5.93	2.77
Sep	0.00	0.00	0.00	33.63	28.98	7.72	70.33	0.00	0.00
Oct	3.17	0.53	3.70	23.45	26.51	2.51	59.88	5.30	0.67
Nov	12.38	2.06	14.44	15.03	24.88	0.00	68.79	18.00	2.53
Dec	3.69	0.61	4.31	24.08	24.31	0.00	57.00	6.48	0.78

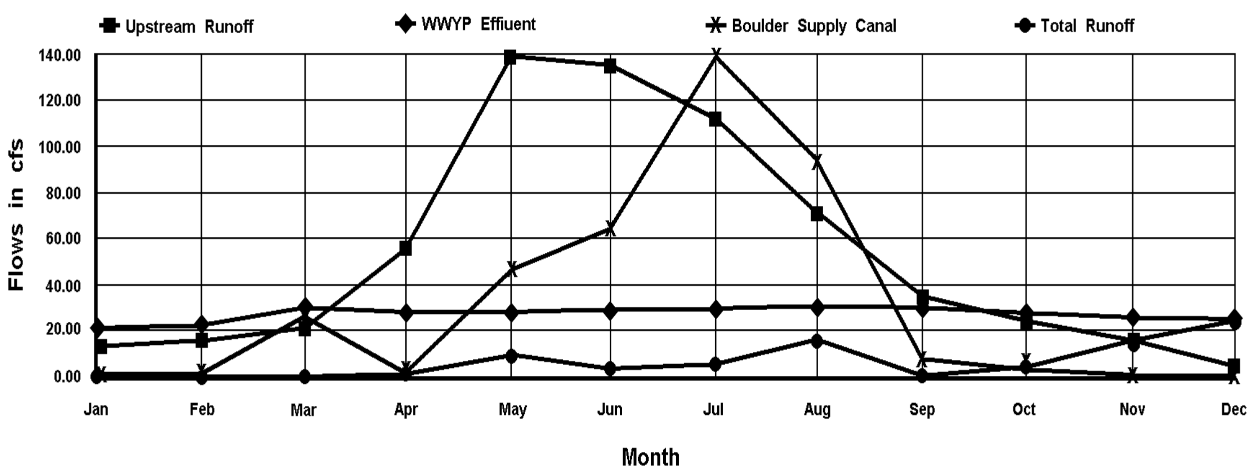


Figure 9-23. Total sources of flow for Boulder Creek, CO, 1992.

Conclusions Drawn from the Water Budget

The results of examining the behavior of Boulder Creek each hour of calendar year 1992 provide dramatic testimony to the influence of man on this stream. Boulder Creek is typical of streams in urban areas because of the intense level of human activities associated with manipulating water resources as part of agricultural, industrial, mining, urban and/or other activities. The following conclusions can be drawn from this water budget:

1. Given the wide variability in flows, even from hour to hour, it is not meaningful to try to find a single "design event" to analyze the impact of urban runoff or any other single term in the water budget.

2. A continuous water budget with a small time step, that is, hourly, is essential in order to capture the reality of stream dynamics.

3. A process oriented approach is essential to accurately characterize what is happening in complex urban stream systems. The Boulder Creek system has evolved over the past 139 years and is a complex combination of facilities and processes including reservoirs, canals, hydropower generation, imports, exports, and instream flow releases. Statistical approaches can be used in conjunction with continuous simulation but a process oriented continuous simulation is essential in order to derive reliable information for risk analysis.

4. A primary purpose of human activities is to reduce the variance in streamflows. The prior appropriations doctrine used in the West allows human activities to be traced and to show how variance reduction occurs due to deliberate human actions.

5. The hydrologic regime changes drastically over the eight mile reach of Boulder Creek as it passes through Boulder. Thus, it is not meaningful to base policy decisions on average conditions. The stream goes from being a rushing mountain stream used for kayaking to a gentle valley stream flowing through open space. Thus, the desirable flow regime varies accordingly.

6. Fish are permanent residents of Boulder Creek. Thus, from their perspective, the flow frequency analysis should be done with a very short time step, say an hour. Existing water quality standards, based on a seven day average low flow, have little meaning to a fish population that has to live in a stream system with flows ranging from 0 to 140 cfs over a single day.

7. The wide variety of stakeholders associated with Boulder Creek continue to adapt the stream system and its management in light of changing attitudes and values. The Boulder Greenways Program, implemented during the past decade, is a dramatic example of these changes as is the City's recently enacted instream flow improvement program.

8. Population and land use management via the open space program have had a major beneficial impact on Boulder Creek. Thus, an integrated appraisal of land and water management is essential.

9. A risk analysis-based approach to the problem can be easily implemented using the results of the continuous simulation model. The frequency distributions need to reflect the appropriate averaging time for the affected species. For Boulder Creek, an hourly time step is essential because of the dynamics of the forcing functions on the system and the short travel times through the system.

Urban Stormwater Quality

Stormwater Pollution in Boulder

The City of Boulder inventoried nonpoint pollution sources within BCW (City of Boulder 1990). Results are summarized here under the headings of agricultural, forest fires, highway, mining and urban runoff.

Agricultural Water Quality

Irrigation using Boulder Creek water is practiced in the lower valley portion of BCW. Irrigation return flows and nonpoint runoff do not have a significant impact on Boulder Creek above 75th St. because this agricultural water enters downstream. Agricultural activities may impact water quality entering Boulder Reservoir.

Forest Fires

A large 4.5 square mile fire occurred during the summer of 1989 in the foothills area called Sugarloaf Mountain. Subsequent heavy rains caused severe soil erosion in the immediate area. Some of these impacts were felt in Boulder Creek with additional sediment accumulations of up to 16 inches.

Highway Runoff

Sanding and salting of highways during the winter months increase loadings to the BCW. Highway 119, which runs parallel to Middle Boulder Creek, is one of the prime concerns due to the relatively heavy traffic and need for extensive ice control due to its mountainous location. During the winter of 1987-1988, a total of 2,869 tons of sand and 201 tons of salt were applied to 17 miles of Highway 119 between Nederland and the canyon mouth. An equivalent amount is applied to county roads that intersect Highway 119. No specific detrimental receiving water impacts have been documented to occur as a result of this activity.

Mining Runoff

BCW was once actively mined. Some residual mine runoff occurs. Gravel mining in the lower portions of BCW has also had an impact on the creek. These problems have been addressed. Some runoff quality problems from mining still exists during relatively wet periods, such as 1995.

Urban Stormwater Quality

Nilsgard (1974) evaluated urban runoff in Boulder. He sampled an urban catchment that drained to Boulder Creek near Broadway. Nilsgard noted the impact of stream diversions on flows in the system. During dry-weather periods, virtually all of the streamflow was diverted at Broadway just above where the storm drain entered Boulder Creek. Base flow in the storm drain provided the only significant dry-weather flow in Boulder Creek at that point. Nilsgard's data showed that urban runoff is equivalent to secondary effluent based on annual loads were calculated. Unfortunately, Nilsgard did not explain how urban loads were calculated. In contrast, analysis completed for this report indicates that urban runoff is much less important than sewage effluent on an annual basis.

Bennett and Linstedt (1978) analyzed Boulder's stormwater quality with a limited sampling program of an urban, suburban, agricultural, and natural area. They sampled six storm events, most of which reflected winter snow conditions. Their results indicate that urbanization appears to cause a decrease in water quality. They did not relate the variable water quality to any beneficial uses. They also looked at treatability. Bennett and Linstedt (1978) concluded that more studies are needed to understand the quality of urban runoff and its impact on the receiving water.

Deacon and Vaught (1993) sampled Boulder Creek upstream of the City (Orodell), in the city (Library and Scott Carpenter Park), and downstream (Valmont). Boulder Creek was sampled in 1991 on April 23, May 30, July 31, September 27, December 6, and on February 4, 1992. All of their results indicate a healthy aquatic environment in Boulder Creek. Unfortunately, they did not describe the flow in the stream nor whether the sampling was related to storm events.

The City of Boulder Stormwater Quality group has been monitoring water quality in Boulder Creek for the past few years. Also, all of the over 1,000 outfalls into the Boulder Creek stream system have been inventoried and checked for dry-weather flows. Generally, Boulder’s stormwater runoff is typical of other urban areas. No significant illicit sources of storm drainage were identified.

Urban stormwater quality can be estimated using event mean concentration estimates, which are based on a national database for the U.S. (Debo and Reese 1994). Also, Denver has collected many samples of urban runoff quality as part of earlier studies of the nature of urban runoff (NURP studies) and more recent NPDES sampling. Boulder has also collected urban runoff quality samples. The national and Denver databases of stormwater samples for suspended solids concentrations were evaluated to see how these concentrations vary both spatially and temporally. A comparison of the means and variances of the two datasets indicates no significant differences in the means or the variances.

The main controls for urban runoff and nonpoint runoff control in Boulder have been a very aggressive land acquisition program, which has set aside about 60,000 acres during the past 25 years. This open space program has the concomitant objective of limiting population growth in the City of Boulder to 160,000 people instead of the earlier projection of 250,000, a 36 % reduction in projected population. Another control is the Tributary Greenway Program wherein the City has acquired riparian lands and created an award winning linear park and greenbelt system, which is heavily used by residents and visitors. A major stream restoration was done as part of this program. The key direct water related component of this study was the City's commitment for instream flow needs with a guaranteed minimum flow of 15 cfs in Middle Boulder Creek as it moves through the City. The City has also installed stormwater detention systems to reduce pollutant loads from some of its tributaries such as Goose Creek. These ponds are an integral part of the Greenway program.

More complete analysis of Boulder's urban runoff quantity and quality is limited by the lack of concurrent measurements of flow and quality from the major storm drains and tributaries. The results of stormwater quality sampling indicate no major problems nor is there any direct evidence of the link between urban runoff and stream impairment, (e.g., fish kills). The City plans to install additional stream gages along Boulder Creek. This will greatly improve the accuracy of estimates of the relative importance of urban runoff.

Recreation and Water Quality in Boulder Creek

Water quality has not been an impediment to recreation in Boulder Creek. The quality is considered to be excellent and much use is made of the stream for kayaking, tubing, and wading. The stream is not used for swimming due to its high velocity, cold temperature, and shallow depths.

Wastewater Characteristics

An important question in analyzing dry and wet-weather quality management strategies is to determine the relative importance of dry- and wet-weather sources. At the most aggregate level, the annual loads from each of these sources can be estimated to obtain the net load after adjusting for removal by treatment. An important question is to characterize the relationship between WWTP flow and concentration. If infiltration and inflow are "pure water," then a straight dilution effect would result.

Brown and Caldwell (1990) present monthly influent data for the Boulder WWTP for the period from CY 1982 to CY 1985. The influent concentration of BOD as a function of WWTP flow are shown in Figure 9-24. The negative relationship shows that concentration decreases as flow increases.

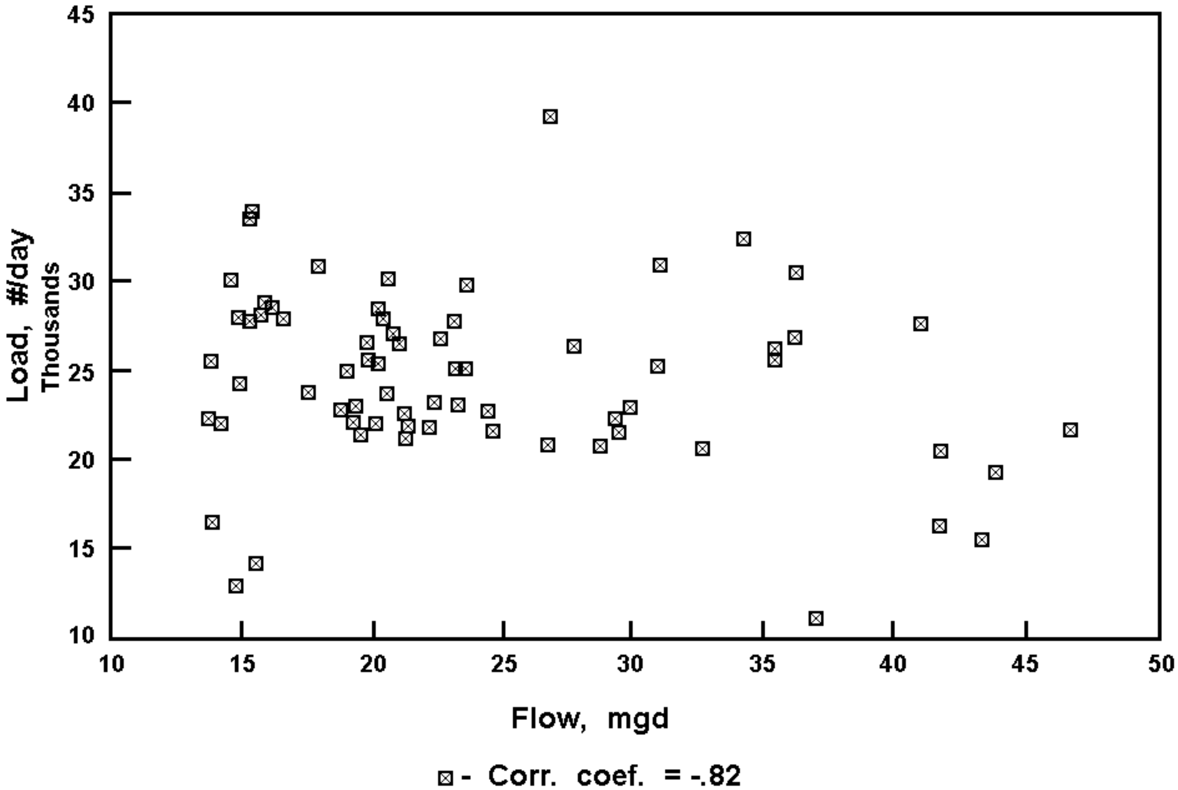
Load as a function of flow is plotted in the upper part of Figure 9-24. The resulting scatter plot indicates that the total load of BOD remains constant at higher flows. This result indicates that, for BOD, a direct dilution effect is occurring. Thus, the added infiltration and inflow are of less concern since they are not causing any significant increase in the BOD load. Figure 9-25, which is a similar plot for SS, reveals a negative correlation but a slight increase in load as flow increases. Thus, the increased flows do cause an increase in the solids load for the WWTP which may cause problems as flows continue to increase.

During the spring of 1995, a major wet weather period occurred with minor flooding and some sewer surcharging. The daily influent flows to the Boulder WWTP from 1990 to June 1995 are shown in Figure 9-26. Influent flows reached over 45 mgd, well beyond any inflows experienced prior to 1995. The WWTP was able to treat all flows without bypassing.

The relationship between WWTP flows and influent quality for BOD are shown in the lower part of Figure 9-24. The concentration decreases sharply as flow increases with influent BOD's dropping from about 250 mg/l at lower flows to less than 50 mg/l at the higher flows. The correlation coefficient for the flow-BOD relationship is -0.82. BOD load as a function of flow during this critical period is shown in the upper part of Figure 9-26. It shows that BOD load remains constant. Thus, the infiltration is simply "clean water" and provides a direct dilution effect.

The results for suspended solids are similar. Figure 9-25 shows the negative correlation coefficient of -0.55 with influent SS concentrations dropping from nearly 300 mg/l to less than 100 mg/l at higher flows. For SS, the loads appear to be constant up to a flow of about 30 mgd. However, beyond 30 mgd, the loads appear to increase significantly, probably as a result of direct inflow of water to the sewers from surface sources.

This negative correlation is of critical importance in evaluating the impacts of wastewater and urban runoff discharges on the receiving water. The negative covariance greatly reduces the potential impact since there is a strong dilution effect as flow increases.



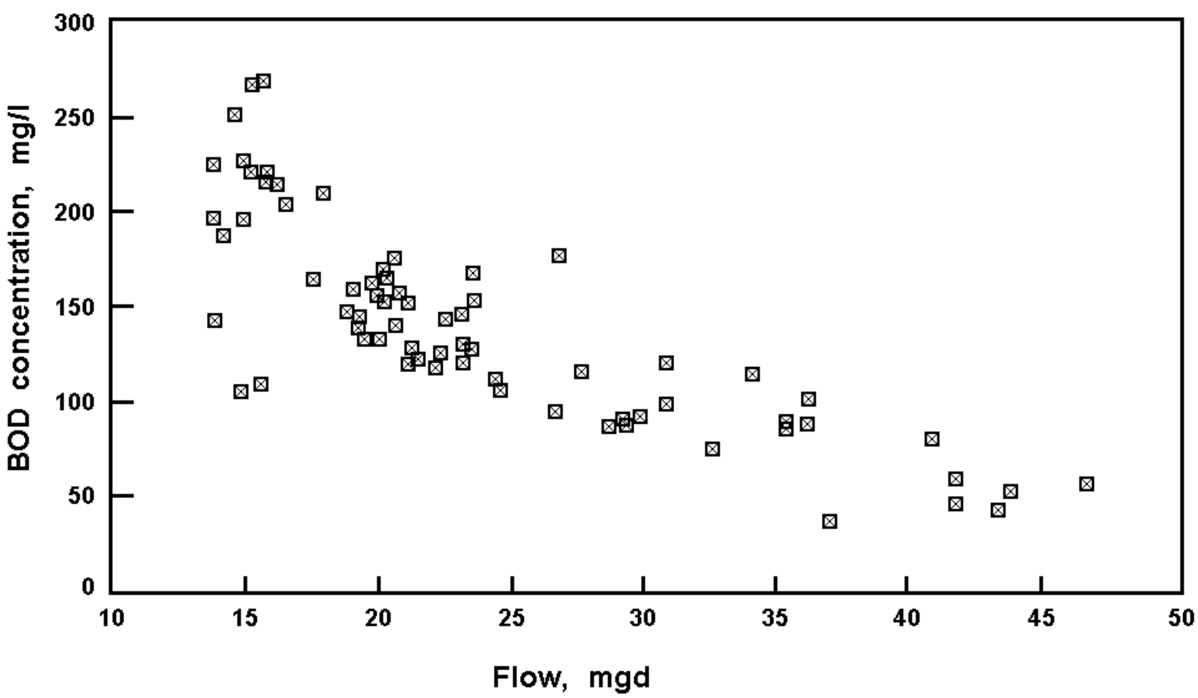


Figure 9-24. Effect of flow on BOD load and concentration, Boulder WWTP, 1990-1995.

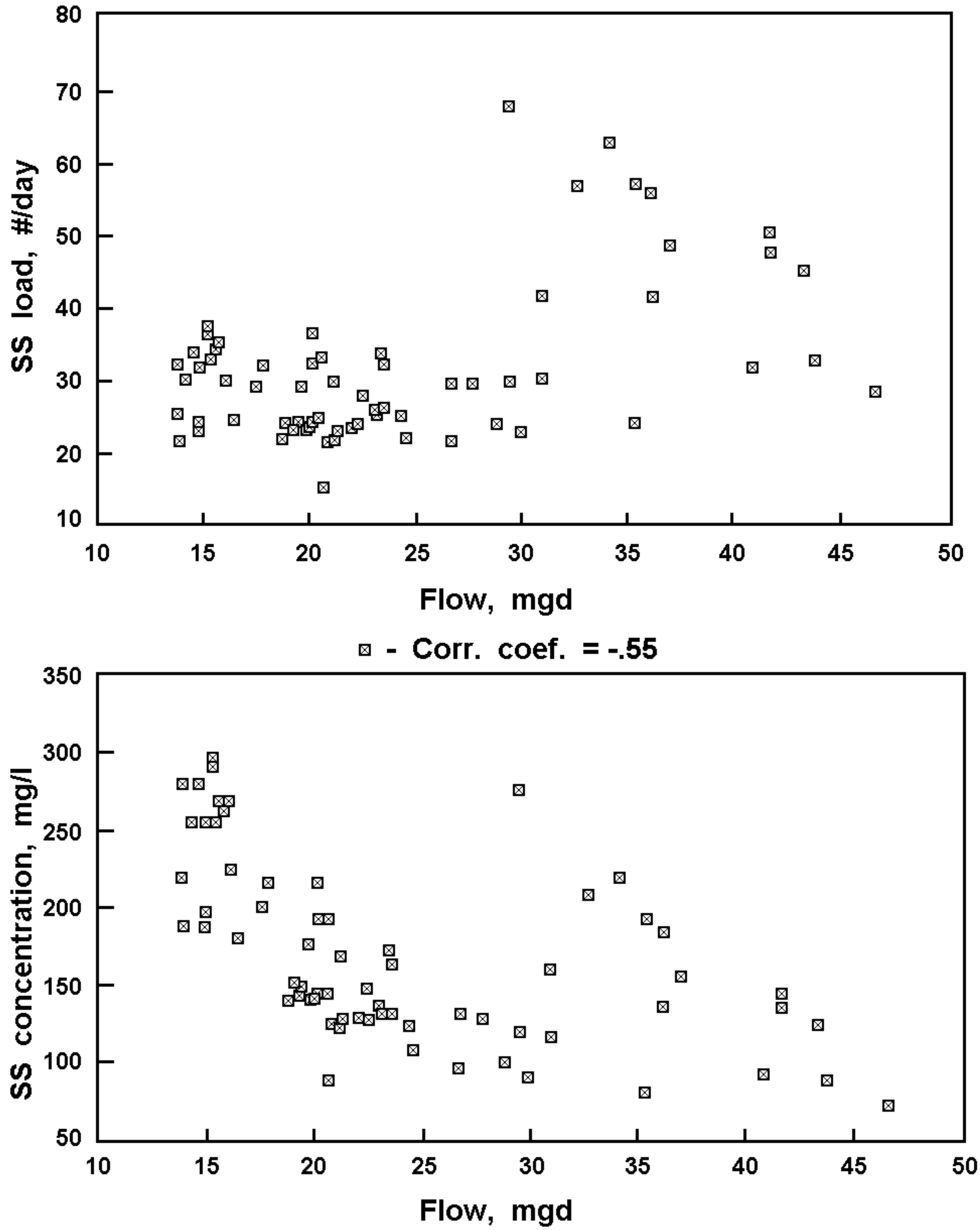


Figure 9-25. Effect of flow on SS load and concentration, Boulder WWTP, 1990-1995.

Removal Efficiencies

The removal efficiencies for the Boulder 75th St. WWTP during 1984 and 1985 were as follows (B&C 1990):

Constituent	Primary	Primary + Secondary
BOD	41%	80%
SS	52%	80%

Removal efficiencies have improved significantly during the past five years as shown in Table 9-14. In 1988, BOD and SS removal efficiencies were about 80 %, the same as the mid-1980s performance. However, since 1989, treatment efficiencies have improved to 1994 removal efficiencies of 93.5% for BOD and 96.6 % for suspended solids, a significant improvement.

Current (1994) variability in treatment plant performance is quite low as shown in Table 9-15. The effluent SS and BOD show very consistent concentrations with coefficients of variation (standard deviation/mean) of about 0.10. Even during the unprecedented wet period of spring 1995, the WWTP produced high quality effluents as shown in Figures 9-27 for SS and Figure 9-28 for BOD. The effluent BOD and SS concentrations are independent of flow rate. Thus, the Boulder WWTP is producing a uniformly high quality effluent with little variability in performance even beyond its nominal design capacity.

Sanitary Sewer Overflows

The City of Boulder has not needed to bypass any of its sanitary sewage, even during the record high flows of spring 1995. This event has a recurrence interval of about one in 25 years. Some localized surcharging of the sanitary sewers did occur for short periods. Thus, Boulder does not presently have a serious sanitary sewer overflow problem.

Overall Receiving Water Quality Impacts

The water quality standards for the State of Colorado classify waters based on the beneficial uses to be protected. The only direct water quality evaluations that have been done are the standard receiving water quality calculations to determine the expected impact of the wastewater treatment plant on Boulder Creek during the one in ten year, seven day duration low flow. This approach to water quality management is extremely narrow because it ignores all of the other components of the water budget and focuses on a single, unusual point in time. As clearly pointed out in the water budget section, the health of the stream is an integration of the continuous impacts over time.

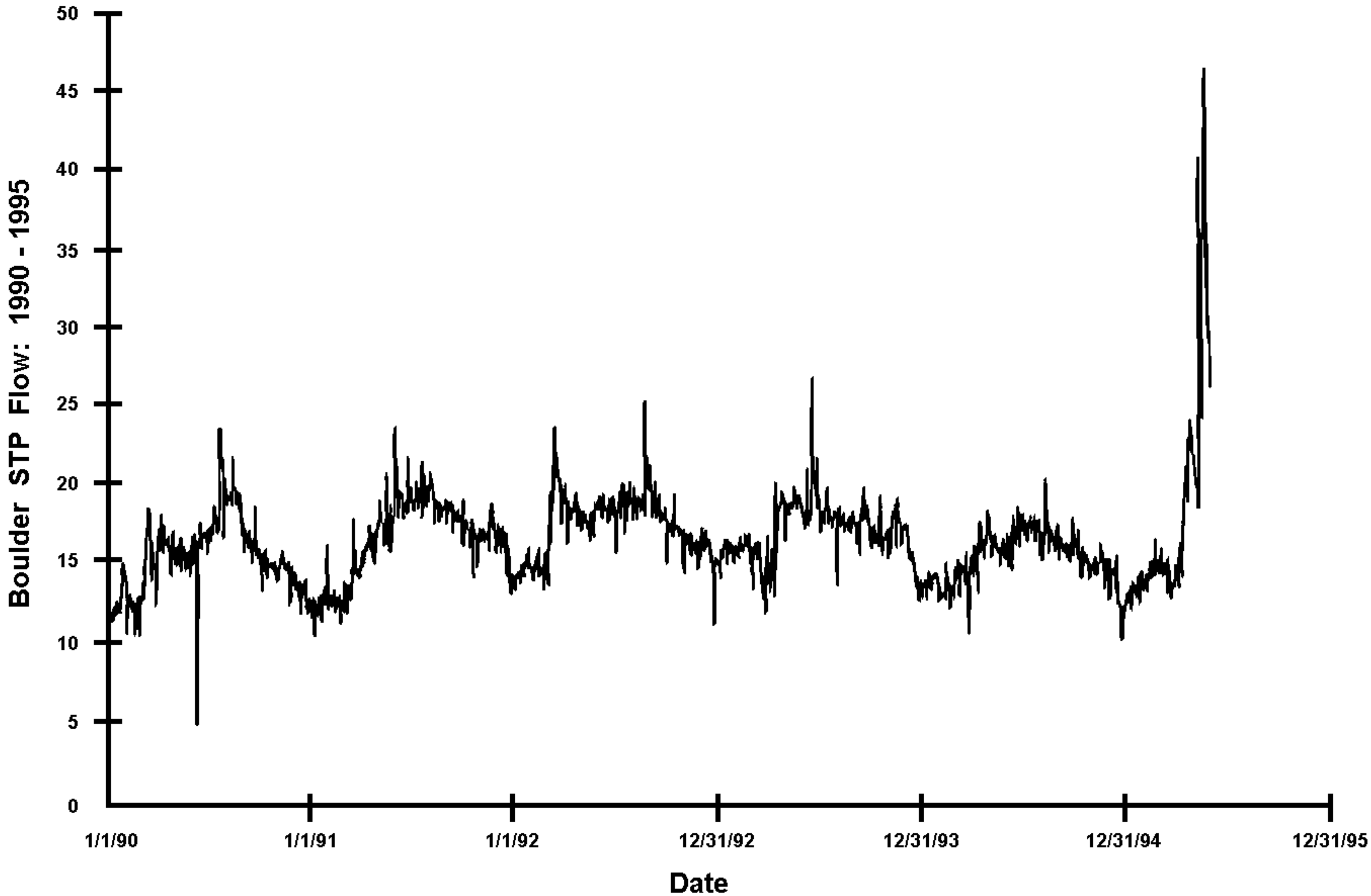


Figure 9-26. Influent flow to Boulder WWTP, 1990 – 1995.

Table 9-14. Trends in annual performance of 75th St WWTP, 1988 – 1994.

Year	Flow (mgd)	Inf. BOD (lb/day)	Inf. BOD (mg/l)	Effl. BOD (lb/day)	Effl. BOD (mg/l)	BOD Removal (%)	Inf. SS (lb/day)	Inf. SS (mg/l)	Effl. SS (lb/day)	Effl. SS (mg/l)	SS Removal (%)
1988	15.4	17036	133	3727	29.02	78.1	16981	132.21	3304	25.72	80.5
1989	15.1	16837	134	2731	21.69	83.8	15838	123.31	1946	15.15	87.7
1990	16.1	19045	142	2332	17.37	87.8	17837	138.88	1048	8.16	94.1
1991	16.5	20064	146	2520	18.31	87.4	18195	141.67	1110	8.64	93.9
1992	17.4	23942	165	1943	13.39	91.9	22635	176.24	1109	8.63	95.1
1993							26268	181	909	6	96.5
1994	15.5	23300	182	1522	10	93.5	24371	189	833	7	96.6
Permit Limit	20.5	29065					29065				

Table 9-15. Trends in monthly performance of 75th St WWTP.

Month-Yr.	Days/mo.	Flow (mgd)	Influent BOD (mg/l)	Influent SS (mg/l)	Effluent BOD (mg/l)	Effluent SS (mg/l)
Jan-92	31	14.5	171	157	24	11
Feb-92	29	14.7	174	158	21	10
Mar-92	31	19.2	134	133	13	8
Apr-92	30	18.3	146	142	13	8
May-92	31	17.8	142	140	13	8
Jun-92	30	18.6	132	126	10	10
Jul-92	31	18.8	139	155	12	8
Aug-92	31	19.3	172	162	10	4
Sep-92	30	18.7	160	166	7	4
Oct-92	31	17.1	202	163	13	7
Nov-92	30	16.1	216	195	16	7
Dec-92	31	15.7	212	183	13	7
Jan-94	31	13.80	194	176	11	6
Feb-94	28	13.40	204	178	13	7
Mar-94	31	14.30	171	159	14	7
Apr-94	30	16.20	164	170	12	6
May-94	31	16.30	137	141	13	6
Jun-94	30	16.80	169	183	11	8
Jul-94	31	17.40	180	166	11	5
Aug-94	31	17.00	183	236	10	6
Sep-94	30	16.40	171	206	12	7
Oct-94	31	15.50	186	202	11	7
Nov-94	30	15.10	206	224	12	8
Dec-94	31	13.30	218	230	12	7
	Statistics for CY 1994					
	Mean	15.46	181.92	189.25	11.83	8.67
	Max	17.40	218.00	236.00	14.00	8.00
	Min	13.30	137.00	141.00	10.00	5.00
	STD	1.39	20.98	28.86	1.07	0.85
	C of V	0.09	0.12	0.15	0.09	0.13

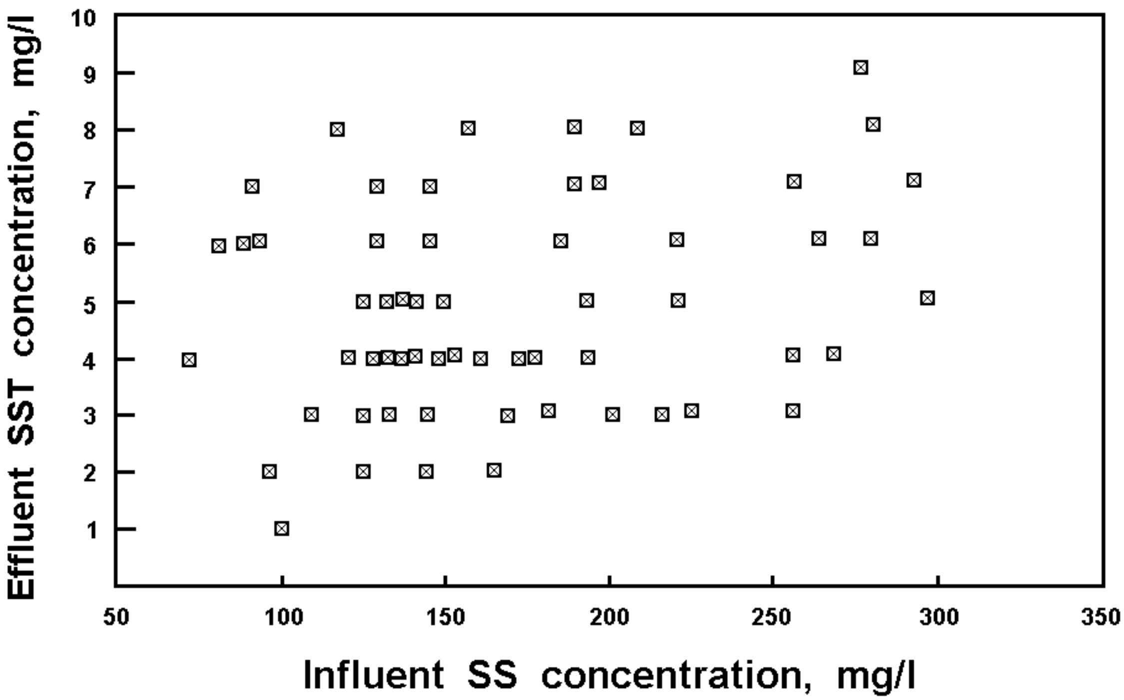


Figure 9-27. Influent vs. effluent SS concentrations, Boulder 75th St WWTP.

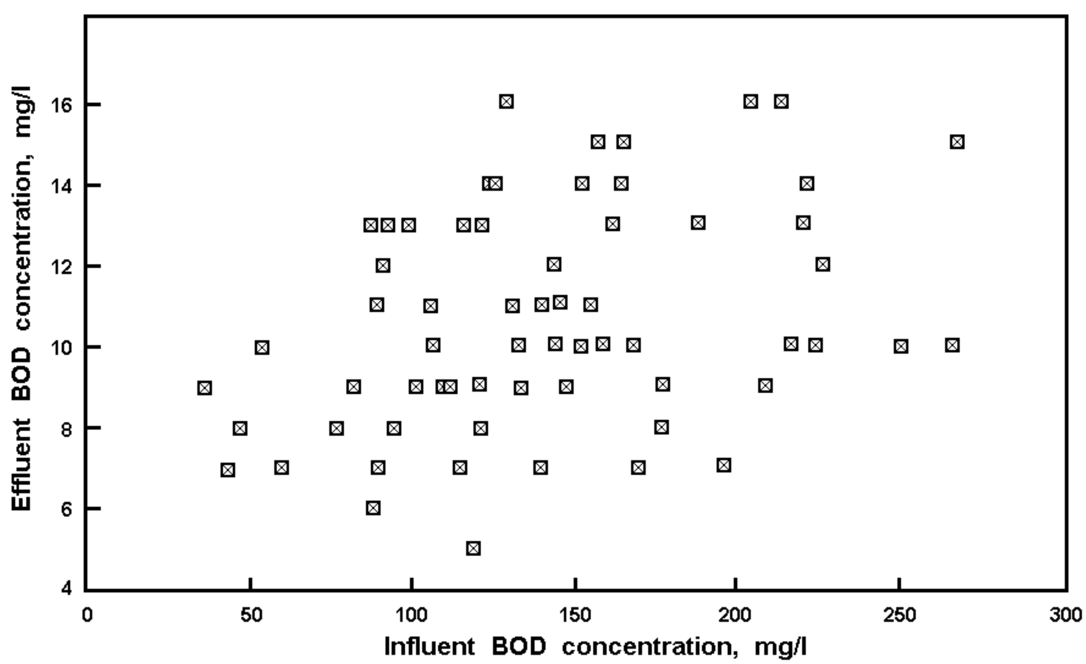


Figure 9-28. Influent vs. effluent BOD concentrations, Boulder 75th St. WWTP.

As pointed out in this case study, BCW is a complex water management system with many competing and complementary uses including water quality management. The eight mile stream section that runs through Boulder goes from a rushing mountain stream to a much slower moving valley stream. Streamflows throughout BC are heavily influenced by human activities. The upper reach is affected by storage and hydropower plant releases. The middle reach is also impacted by heavy diversions during the warmer months of the year. Lastly, the lower reach receives a major increase in flow due to water imports and the return flow from the WWTP. The potential impacts of stormwater quality on Boulder Creek are discussed here for the upper, middle, and lower sections of the creek.

Upper Section-Boulder Creek Immediately Above the City

This section of the creek does not receive any significant urban runoff. The upstream land uses are almost all natural since the land is publicly owned and managed either by the U.S. Forest Service or the City or County of Boulder. Thus, the runoff quality is excellent. Urban runoff quality does not affect this section. The major impact on this section is the upstream diversions and pulsing of flows that reduce the quantity of flow and increase the hourly variability of flows. This section of the stream is used for kayaking and was the site of the 1995 Olympic Festival kayaking competition.

Middle Section-Boulder Creek at 28th St.

This section of the creek receives urban runoff from the immediately surrounding drainage area. The concentration of this urban runoff would be typical of the reported values in the literature. Only about 20% of Boulder's urban runoff enters the middle part of the stream. This runoff is diluted by runoff from adjacent open space lands. Thus, the volume of urban runoff is relatively small. The major impact in this middle section is the greatly reduced flows in the stream because of upstream diversions as the water enters the city. Thus, less dilution water is available. The City has implemented a major program to augment these low flows and the stream has undergone restoration as part of the Greenways Program. No significant urban runoff quality problems have been reported for this reach. Intensive use is made of this section of the creek because of the creation of a Greenway about ten years ago. Current activity levels exceed one million people per year. The stream restoration recently won a national award.

Lower Section-Boulder Creek Below 75th St.

This section receives all of the urban runoff from Boulder. Some of this urban runoff has received treatment in detention systems, (e.g., Goose Creek). It also receives the return flow from the Wastewater Treatment Plant and imported water from the Colorado-Big Thompson Project. Urban runoff is a relatively small source of water, less than 25% of the WWTP effluent and only 20% of the Colorado-Big Thompson imported water. The WWTP provides a consistently excellent effluent quality even during very high flow periods such as the spring of 1995. The most sensitive time of the year for this section is early fall after the imports have ceased and when the upstream flow is low. This section of the stream is not presently accessible to the public. Thus, there is little recreational activity to report.

Risk-Based Analysis of Urban Runoff Quality

The mixed concentration of a constituent in a stream can be calculated as follows:

$$Co = (CsQs + CrQr)/(Qs + Qr) \qquad \text{Equation 9-1}$$

where *Co* = downstream concentration, mg/l,
Cs = upstream concentration, mg/l,
Cr = concentration of added inflow, mg/l,
Qs = upstream flow, and
Qr = added inflow.

The added inflow can be of several types including:

1. Direct urban runoff.
2. Sanitary sewer overflow.
3. Wastewater effluent.
4. Imported water.

For Boulder Creek, direct urban runoff occurs at numerous places along the stream. There are no sanitary sewer overflows. Wastewater effluent enters the stream downstream of the City as does the imported water from the Colorado-Big Thompson Project.

Analysis of the terms in Equation 9-1 and there statistical properties is critical to understanding the stream water quality impacts. The key factor which has been neglected in the literature is the covariance of concentration and flow. Covariance is defined as:

$$s(xy) = (x-xb)(y-yb) \qquad \text{Equation 9-2}$$

where *s(xy)* = covariance between x and y,
x,y = two variables, and
xb, yb = means of x and y.

The correlation coefficient measures the extent of the covariance, or

$$r(xy) = [(x-xb)(y-yb)]/[(x-xb)^2*(y-yb)^2] \qquad \text{Equation 9-3}$$

where *r(xy)* = correlation coefficient between x and y with
$-1 <= r <= +1$.

The expected covariance patterns for the terms in Equation 9-1 are discussed in the following:

Covariance Between Concentration and Flow

For urban runoff, if a finite amount of material is on the land surface, say a parking lot, then one would expect to see a negative covariance between concentration and flow. However, if the source of material is large, say suspended solids from a construction area, then one could indeed see a positive covariance. For most constituents, a negative covariance between concentration and flow would be expected as was observed for the WWTP influent. This negative covariance reduces the expected impacts of stormwater runoff since a dilution effect occurs.

Covariance Between Upstream Flow and Urban Runoff

The following statistics on causes of 1994 beach closings in the U.S. were reported (Water Environment and Technology 1995):

Cause	Number
Sanitary Sewer Overflows	584
Stormwater Runoff	345
Combined Sewer Overflows	194
Agricultural Runoff	136
Wastewater Treatment Plant Malfunctions	106

While beach closings is not an issue for Boulder Creek, the above statistics do give some indication of the relative importance of the various wet-weather sources and WWTP malfunctions. In the case of oceans or large lakes, the covariance between the stormwater runoff and the receiving water capacity would be expected to be zero. However, for riverine systems, one would expect it to be positive, that is, when urban runoff is entering the stream, the flow in the stream is increasing due to runoff from upstream concurrently entering the system. For Boulder Creek and the City of Boulder, the following combinations of wet-weather scenarios occur.

1. Worst Case: Localized rainfall over developed portion of the urban area only. Low base flow in the stream. This situation can occur in late summer. Thus, upstream flows would be low and most of the stream runoff would be urban runoff. This situation would be expected to happen a few times a year associated with light storms.

2. Typical Case: Moderate basin wide rainfall and runoff. This situation would be associated with the more significant storm events. In this case, the urban runoff would be a small part of the total runoff since only about 7% of the land use in BCW is urban land use.

3. Significant Wet-Weather Events: Significant wet-weather events occur one to five times per year. These events include the major flooding events, which are rarer. Under this scenario, all of BCW would be expected to be contributing flow

and infiltration entering the WWTP would be expected to be relatively high due to the wet conditions. In this case, urban runoff would be an insignificant part of the streamflow and water quality load.

Ideally, the probability density function for all of these scenarios can be developed. However, insufficient data were available to make these judgments. It is possible to show the covariance of streamflow and wastewater treatment plant flow. A total of 526 wetter days from 1990 to mid 1995 were analyzed to compare the flow in the WWTP with the flow in Boulder Creek immediately upstream of the WWTP and the imported water from the Colorado-Big Thompson project. The results, shown in Figure 9-29, indicate a strong positive covariance of streamflow and WWTP flow. The correlation coefficient is +0.81.

This covariance plot has significant implications for evaluating the impact of WWTP bypasses or overflows during wet-weather periods. Current thinking is that CSO or SSO should not occur more than a few (one to five) times per year. Thus, the system would capture and treat all of the moderate storms. During the larger storms, part, not all, of the larger events would be bypassed. How serious is this problem? If the covariance between wastewater flows and receiving water flows is determined, then one could conclude that the CSO and SSO volume is an insignificant part of the stream runoff during this very wet period.

Thus, a relatively complex combination of the joint probabilities of undesirable conditions may occur. This situation can be estimated with reliable continuous simulation or Monte Carlo analysis. The results shown in Figure 9-29 indicate 23 days when the flow in the WWTP was at least 40 cfs. This would correspond to about four events per year, well within the current guidelines of the allowable number of overflows per year. But according to the covariance analysis, if the WWTP flow is 40 cfs, then the Boulder Creek flow would be over 500 cfs, or a dilution ratio of over 14:1. At a WWTP flow of 70 cfs, the expected flow in Boulder Creek would be over 1600 cfs, a dilution ratio of over 23:1. This assumes that all of the storm is bypassed. In reality, only part of the storm would be bypassed. If the capacity of the plant was 50 cfs, then the bypass would be the difference. Thus, the expected overflow for the 70 cfs case is 20 cfs, not the entire 70 cfs. Correspondingly, the dilution ratio is about 80:1.

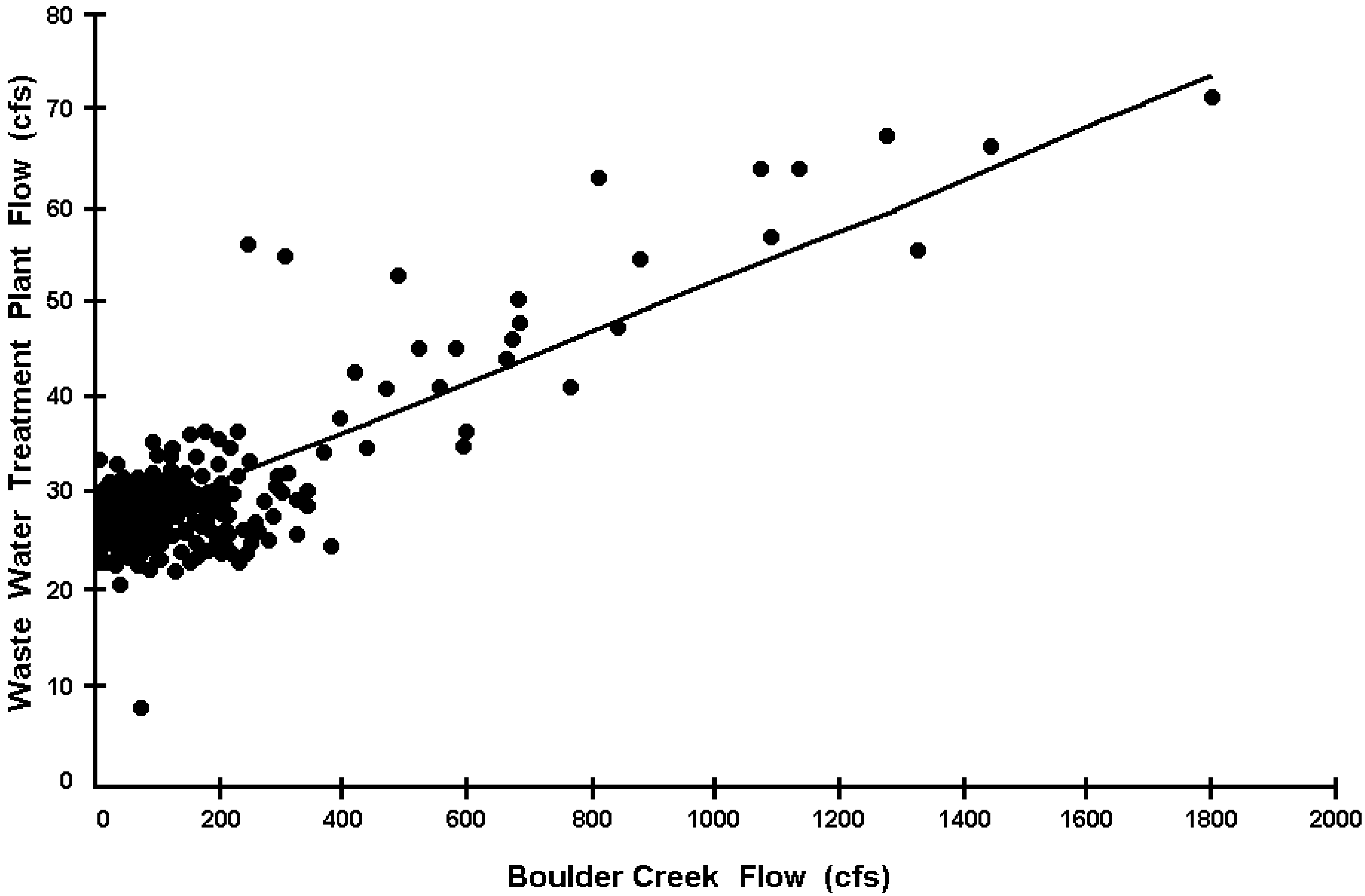


Figure 9-29. Boulder WWTP flow vs. flow in Boulder Creek.

The key point brought out by the risk analysis is that including the covariance among concentration and flow and among flows is critical. All of these covariances help reduce the impact of stormwater runoff. Negative covariance between concentration and flow indicates that the concentrations decrease at higher flows. The positive covariance between upstream flows and wastewater flows means that significant dilution capacity is available during these wetter events. Also, overflow events do not bypass all of the event, but only part of it. Thus, the impacts are even lower.

Ultimately, real-time water management will exist in urban areas. Thus, cities will be able to deterministically manage the concentrations and the flows entering the receiving waters throughout the year. The City of Boulder may have this capability in the next five to 10 years. This real-time control will reduce the probability of "worst case" conditions occurring since the system can be managed to avoid these possibilities.

Overall, the benefit-cost-risk perspective provides valuable insights into the urban stormwater quality problem and to evaluating urban water systems in general. A key ingredient of improved water management is direct measurement of the behavior of the system and the management flexibility to take advantage of multipurpose water and land management opportunities. The City of Boulder and BCW offer numerous illustrations of the benefits of this approach.

References

Anonymous (1992). Hydata. 11, 6.

Bennett, E.R. and K.D. Linstedt (1978). Pollutional Characteristics of Stormwater Runoff. Colorado Water Resources Research Institute Completion Report No. 84. Fort Collins, CO.

Brown and Caldwell (1990). Treated Water Master Plan, Phase 1, Final Report. City of Boulder.

City of Boulder (1983). Boulder Reservoir-Development Master Plan. Boulder, CO.

City of Boulder (1990). Boulder Creek Basin Planning to Reduce Nonpoint Pollution by Using Best Management Practices. Boulder, CO.

City of Boulder, (1998) Boulder Creek Watershed, CO. Hydrosphere, Boulder CO.

City of Boulder, (1998). Long Range Management Policies. City of Boulder Open Space GIS Lab, R Grover, Boulder, CO,.

City of Boulder Planning Department and Boulder County Land Use Department (1990).

Deacon, J.R. and D.G. Vaught (1993). Assessment of Water Quality of Boulder Creek, Boulder County, CO. Analysis of Water and Sediment Chemistry and Benthic Invertebrate Communities. Research Project. U. of Colorado at Denver.

Debo, T. N. and A.J. Reese (1995). Municipal Storm Water Management. Lewis Publishers. Boca Raton, FL.

Heaney, J.P. (1993). New Directions in Water Resources Planning and Management. Water Resources Update. Fall.

Lacy, G. (1995). Personal communication with the Director of the Stream Restoration Program. City of Boulder, CO.

Loucks, D.P. (1995). Developing and Implementing Decision Support Systems: A Critique and a Challenge. Water Resources Bulletin. Vol. 31, No. 4.

Maass, A., et al. (1962). Design of Water Resource Systems. Harvard University Press. Cambridge, MA.

Mays, L.W. and Y-K. Tung (1993). Hydrosystems Engineering and Management. McGraw-Hill. New York, NY.

McPherson, M.B. (1973). Need for metropolitan water balance inventories. Jour. of the Hydraulics Div. ASCE. 99, HY10, p. 1837-1848.

Nilsgard, V. (1974). A Characterization of Urban Stormwater Runoff in Boulder, Colorado. MS Thesis. Dept. of Civil, Environmental, and Architectural Engineering. U. of Colorado. Boulder, CO.

Office of Technology Assessment (1982). Use of Models for Water Resources Management, Planning, and Policy. Congress of the United States. Washington, D.C.

Peterson, M.S. (1984). Water Resource Planning and Development. Prentice-Hall. Englewood Cliffs, NJ. 316 p.

Rozaklis, L. (1994). Boulder Creek instream flow program. Talk at U. of Colorado at Boulder.

Smith, D.S. and P.C. Hellmund, (Eds.) (1993). Ecology of Greenways. U. of Minnesota Press. Minneapolis, MN.

Smith, P. (1987). History of Flooding in Boulder. City of Boulder, CO.

USEPA (1991). The Watershed Protection Approach, An Overview. Office of Water. U.S. Environmental Protection Agency. Washington, D.C. EPA/503/9-92/002.

Viessman, W., Jr. and C. Welty (1985). Water Management: Technology and Institutions. Harper and Row Publishers. New York, NY.

WBLA (1988). Raw Water Master Plan. City of Boulder. Boulder, CO.

WEF (1993). Proceedings Watershed '93. WEF. Alexandria, VA.

WEF (1996). Proceedings Watershed '96. WEF. Alexandria, VA.

Wurbs, R.A. (1994). Computer Models for Water Resources Planning and Management. IWR Report 94-NDS-7. U.S. Army Corps of Engineers. Alexandria, VA.

Chapter 10

Cost Analysis and Financing of Urban Water Infrastructure

James P. Heaney, David Sample, and Len Wright

Introduction

The purpose of this chapter is to provide summary information regarding the cost of water, wastewater, and stormwater infrastructure for U.S. cities. While the main theme of this report is stormwater, some of the innovative ideas proposed relate to water supply. An example is reusing stormwater for irrigation to reduce water supply demands.

Demand for Water Infrastructure

The effect of dwelling unit (DU) density on water use is shown in Table 10-1, on wastewater is shown in Table 10-2, and on stormwater is shown in Table 10-3. The wastewater table uses the indoor water supply as the estimate for base wastewater flows. A range from two to 10 DU's per gross acre is used since most residential developments fall within this range. Gross area is defined as the lot and the right-of-way in the neighborhood only. It does not include open space or other land uses. The procedure and the results are described next for the three components of urban water systems.

Effect of Density on Imperviousness

The effect of DU per acre on pervious and impervious areas was evaluated using the database described in Chapter 3. The square feet of land devoted to pervious and impervious areas, as a function of DU per acre, is shown in Figure 10-1. At two DU's per acre, the total land area is about 21,800 square feet. About 12,000 square feet of this land is pervious. At the other end of the scale, only 1,600 square feet of pervious area exists for a density of 10 DU's per acre. The difference in pervious area per DU is dramatic, even over this relatively small range of DU densities. Similarly, the impervious area increases from about 2,750 square feet at 10 DU per acre to 9,800 square feet per acre at two DU per acre, over a three-fold increase. Thus, even though the percent imperviousness decreases as density decreases, the total imperviousness per DU increases significantly.

Effect of Density on Pipe Length

Using the same database, the effect of density on lot width is shown in Figure 10-2. Between three and 10 dwelling units per acre, the lot width varies linearly ranging from 25 feet at 10 DU per acre to 90 feet at three DU per acre. Below three DU per acre, the lot width increases at a more rapid rate, reaching 140 feet at two DU/acre.

Table 10-1. Effect of dwelling unit density and irrigation rate on indoor and outdoor water use.

Percent of irrigable area that is watered:			75%					
	Irrigation rate (inches/yr.):		5	10	15	20	30	40
Dwelling Unit Density (DU/acre)	Pervious Area (sq. ft./DU)	Indoor[1] Daily Use (gal./DU)	Annual average irrigation (gal./DU)					
2	14,000	180	77	154	231	307	461	615
4	5,500	180	40	79	119	159	238	318
6	3,100	180	22	45	67	90	134	179
8	1,900	180	13	26	38	51	77	102
10	1,400	180	10	20	31	41	61	82

1) Assumed indoor water use in gallons per capita per day =60
Assumed number of people per dwelling unit =3

Table 10-2. Effect of dwelling unit density on wastewater and infiltration/inflow.

Dwelling Units Density (DU/acre)	Indoor[1] Daily Use (gal./DU)	Lot Width Or Frontage (ft./DU)	Assigned[2] Feet of Pipe (DU)	I/I[3] Daily (gal./DU)
2	180	140	70	350
4	180	82	41	205
6	180	62	31	155
8	180	42	21	105
10	180	22	11	55

1) Base wastewater flow is assumed to equal indoor water use from previous table.
2) Feet of pipe per dwelling unit is 0.5*feet of frontage per dwelling unit.
3) Assumed infiltration/inflow rate in gallons/day/foot = 5

Table 10-3. Effect of dwelling unit density and runoff rates on quantities of stormwater runoff.

Runoff from impervious area (inches/yr.):			10	20	30	40
Dwelling Units Density (DU/acre)	Indoor Daily Use (gal./DU)	Impervious Surface (sq. ft/DU)	Daily Runoff (gal./DU)	Daily Runoff (gal./DU)	Daily Runoff (gal./DU)	Daily Runoff (gal./DU)
2	180	9,780	167	334	501	668
4	180	4,690	80	160	240	320
6	180	3,760	64	128	193	257
8	180	3,445	59	118	176	235
10	180	2,756	47	94	141	188

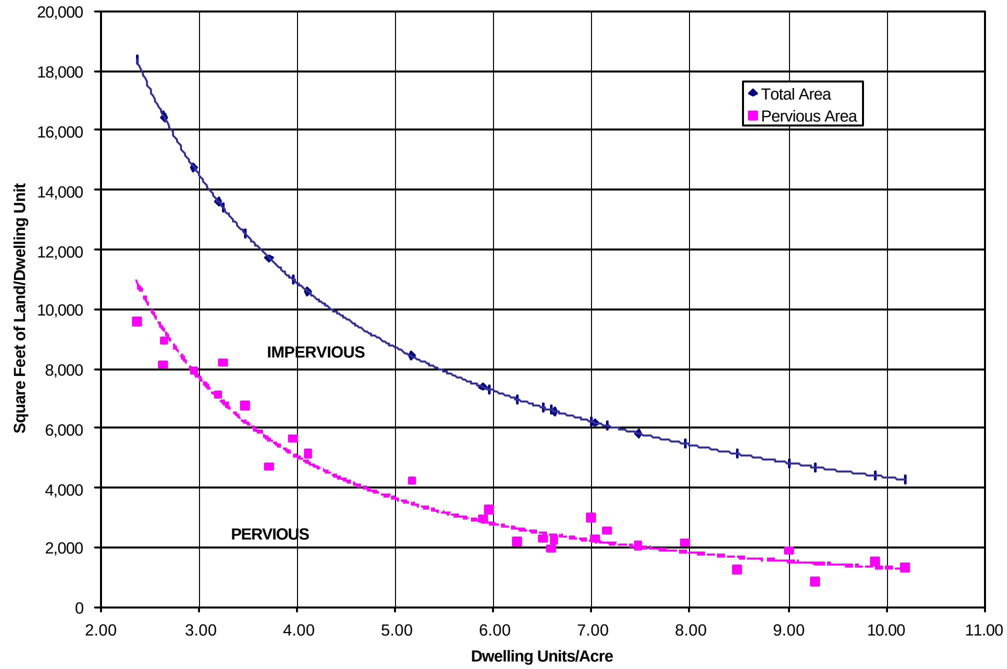


Figure 10-1. Pervious and impervious area as a function of dwelling unit density.

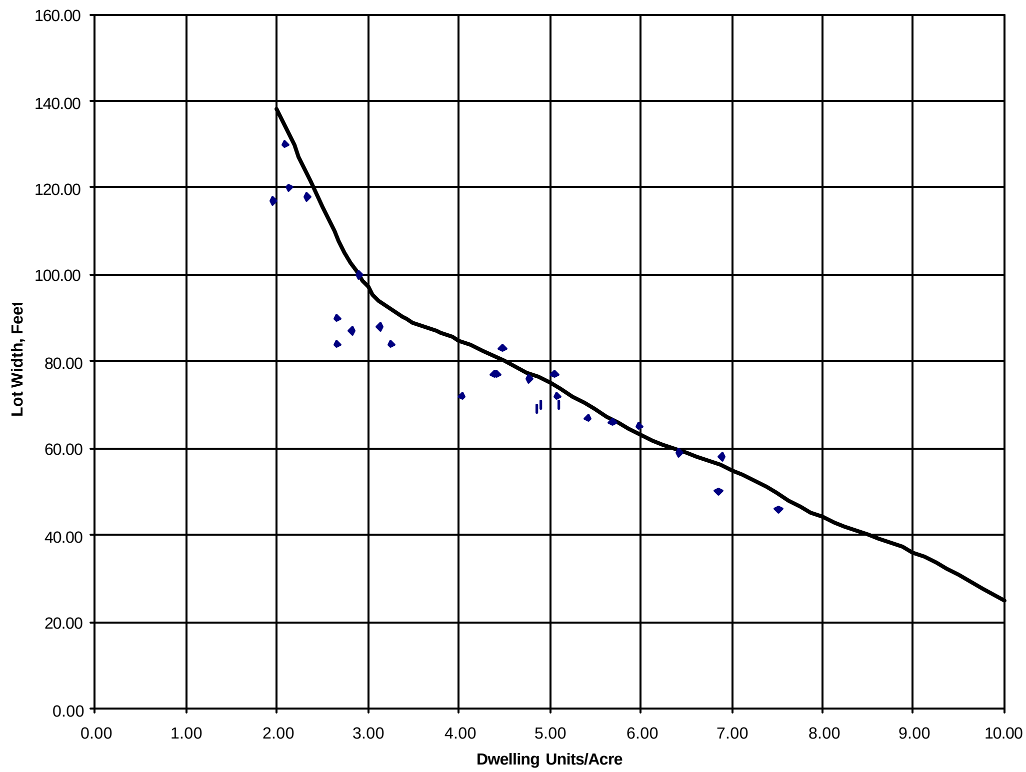


Figure 10-2. Lot width as a function of dwelling unit density.

The total pipe length required to serve a given customer is the sum of the length immediately in front of the property and a prorated share of the pipes in the system that serve multiple users. The mix of pipes depends on the nature of the network and the size of the system. The best general databases found on the network hierarchies, for purposes of this report, were for sanitary sewers and street networks. Dames and Moore (1978) conducted a national survey of 455 sewer construction projects. The final results for sanitary sewer pipe lengths and diameters arranged by population size groups, are presented in Table 10-4.

If local pipes are assumed to be 14 inches or less, then the ratio of large pipes to small pipes can be determined as shown in the last column of Table 10-4. These ratios are plotted as a function of population served in Figure 10-3. The ratios are seen to increase from about 0.15 for a small system serving about 1,000 people to about 0.4 for systems serving a population of 400,000.

Another measure of the reasonableness of the preceding ratio is obtained by looking at the urban street systems having a geometry similar to pipe networks. The results of a 1995 national summary of urban streets is presented in Table 10-5. The ratio of larger roads to local roads is 0.44 and the ratio of larger roads to collector and local roads is 0.25.

Lastly, an inventory of the water pipe network for Boulder, CO, shown in Table 10-6, indicates ratios ranging from 0.17 to 0.41 depending upon how "small" is defined. Boulder is a city of about 100,000. These comparative ratios for streets and water mains indicate that the ratios based on the Dames and Moore study are reasonable.

Table 10-4. Sanitary sewer pipe in place for various city sizes (Dames and Moore 1978).

Population Range		**Mileage of Various Pipe Sizes**					**Feet of larger pipe/feet of Smaller pipe[1]**
From	**To**	**<8"**	**8"-14"**	**15"-24"**	**> 24"**	**Total**	
500,000	>	1,094	39,649	14,971	12,646	68,360	0.68[2]
250,000	500,000	4,860	26,123	7,420	4,990	43,393	0.40
100,000	250,000	5,010	34,824	5,662	4,610	50,106	0.26
50,000	100,000	10,061	29,925	6,108	5,236	51,330	0.28
25,000	50,000	9,233	34,609	6,749	3,402	53,993	0.23
10,000	25,000	19,041	47,946	7,264	2,218	76,469	0.14
2,500	10,000	23,987	74,257	12,740	3,787	114,771	0.17

1) Assume neighborhood pipes are 14" in diameter or less. These pipes are considered to be "small".
2) Sample calculation: (14,971+12,646)/(39,649) = 0.68

Table 10-5. Street mileage in the U.S. - 1995.

Urban	Miles of road	% of urban
Interstate	13,307	1.6%
Other freeways/expressways	9,022	1.1%
Other principal arterial	53,044	6.4%
Minor arterial	89,013	10.8%
Collector	87,918	10.6%
Local	574,119	69.5%
Total Urban	826,423	100.0%
Total Rural	3,100,301	

Source: STAT: State Transportation Analysis Tables, (http://www.bts.gov/cgi-bin/stat/final_out.pl)

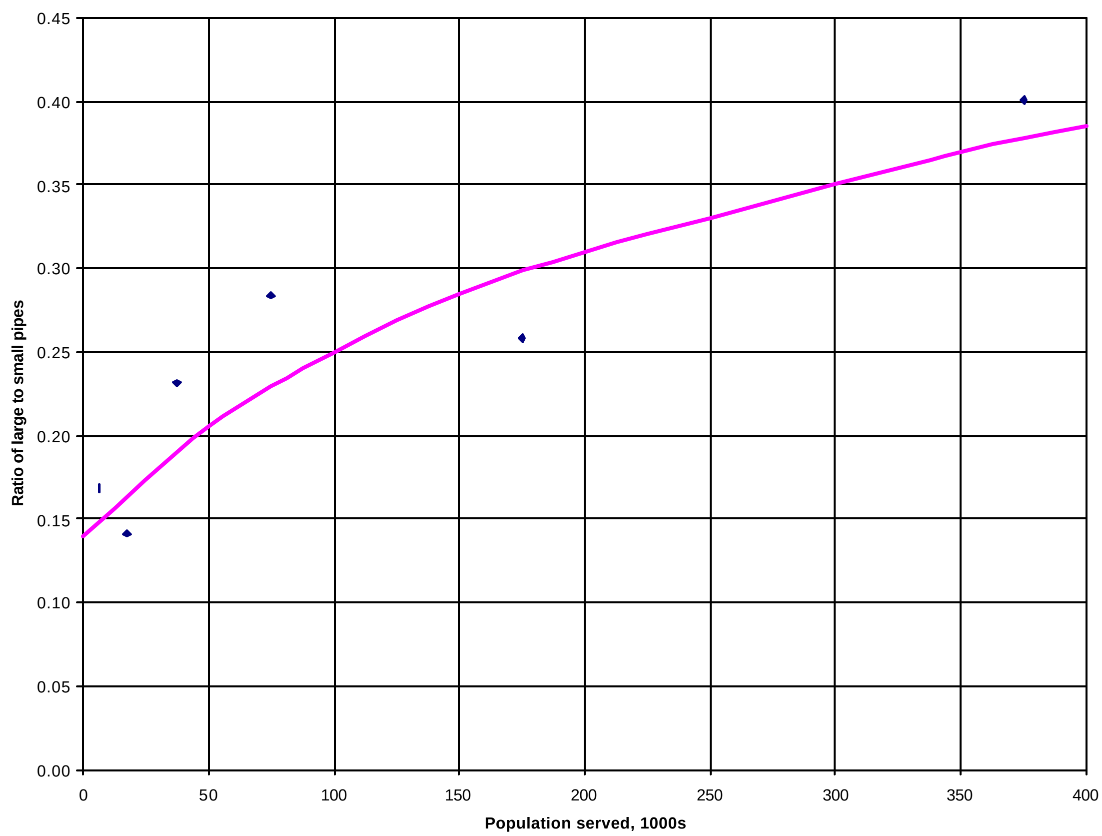


Figure 10-3. Effect of population on the ratio of length of large pipes to length of small pipes.

Table 10-6. Summary of water pipe diameters and lengths in Boulder, CO.

Diameter (inches)	Length, (1000 ft.)	Cumulative Length, (1000 ft.)	Cumulative %
4	107	107	5.3%
6	517	624	31.0%
8	806	1430	71.0%
10	1	1431	71.1%
12	288	1719	85.4%
14	14	1733	6.0%
16	132	1865	92.6%
18	19	1884	93.5%
20	35	1919	95.3%
24	59	1978	98.2%
26	2	1980	98.3%
30	34	2014	100.0%
Total	2,014		

Assume that all pipes <= 12" serve neighborhood systems

Length of smaller pipes in feet: 1431
Length of larger pipes in feet: 583

Ft. of larger pipe/ft. of smaller pipe = 0.41
If 12" is "small," the multiplier is 0.17
If 12" is "large," the multiplier is 0.41
Use average of 0.29

Water Supply

Based on the recent North American End Use Study (NAREUS) described in Chapter 3, an average of 60 gpcd is used for indoor water use. Also, the assumed population per dwelling unit is three persons, based on the NAREUS results. Indoor water use per DU is independent of lot or house size.

Outdoor water use was estimated as a function of the pervious area. About 75% of the pervious area is assumed to be the potentially irrigable area. The water budget presented in Chapter 8 provides detailed information on the expected water deficits for various cities in the United States. Based on calibration data for Denver, the deficits shown in Table 8-3 should be doubled to reflect actual practice. Key reasons for the differences include the fact that not much of the precipitation is viewed as being "effective" by users. Also, they may over irrigate (Stadjuhar 1997). The resulting water use in gallons/DU as a function of the irrigation rate in inches per year was shown in Table 10-1.

For a given irrigation rate, say 15 inches per year, which is similar to Denver practice, the daily irrigation use exceeds the indoor water use at lower population densities. On the other hand, at DU densities greater than six, the outdoor water use remains less than the indoor water use even for high irrigation rates. The key factor that affects urban water supply systems is the strong trend towards lower DU density and the corresponding large increase in pervious area per DU. Thus, even with improved water conservation practices, outdoor water demand has been increasing due to the lower population densities associated with urban sprawl.

Wastewater

The base wastewater flow can be estimated as the indoor water use. The main source of uncertainty in wastewater flows is the amount of I/I. While I/I is a complex process, most predictive models use feet of sewer as a key explanatory variable. For this case, a rate of five gallons per day per foot of pipe is used. The resulting sewer flows, shown in Table 10-2, indicate that I/I exceeds base wastewater flow as the population density decreases below about five DU/acre. If the effect of population on pipe length per DU is included, then the dominance of I/I becomes even more apparent. Of course, all of these conclusions assume a constant I/I rate of five gallons per day per foot of pipe.

Stormwater

Stormwater runoff rates depend on local precipitation patterns and the extent of imperviousness. As shown in Table 10-3, the impervious area per DU increases almost by a factor of four as density decreases from 10 to two DU per acre. Thus, even though the percent imperviousness might decrease, the total impervious area increases greatly as densities decrease. For lower densities, the annual quantities of stormwater exceed indoor water use for most parts of the country. In addition, if storage of the first half inch of runoff is required, then the storage area per DU increases significantly as densities decrease. The feet of drainage pipe per DU can be estimated as a function of the lengths calculated above for sanitary sewers. The length of storm sewer required per DU would be less than for sanitary sewers in the more arid areas since overland flow on the street can be used instead of pipes for some of the local travel.

Optimal Scale of the Urban Water System

The regionalization problem addresses the tradeoff between the economies of scale of the treatment plant, and the spatial diseconomies of scale of pipeline distances, as distances become large. For a description of this problem, the reader is referred to Heaney (1997), Whitlach (1997), and Mays and Tung (1992).

Adams, Dajani and Gemmell (1972) evaluated the optimal size of service area for wastewater collection and treatment systems. They show that the collection systems exhibit diseconomies of scale because of the increasing lengths of pipe per unit of flow while treatment plants exhibit economies of scale. Their results, presented in Figure 10-4, show that the optimal size of wastewater service area decreases as population density decreases and that the diseconomy is quite significant if one exceeds this size service area.

The lowest population density shown in this figure is 15 persons per acre, or about four to five DU per acre. Sprawl is considered to occur at densities less than three units per acre. These results strongly suggest that the optimal size wastewater service area for contemporary low density developments is well within the neighborhood size suggested in this report. Also, Adams, Dajani and Gemmell (1972) argue that decentralized wastewater systems can provide better water quality than highly centralized systems because they make better use of the assimilative capacity of the receiving water and average out stochastic fluctuations in the performance of individual plants.

Clark (1997b) evaluates the effect of size on the least cost combination of collection and treatment using data collected for the City of Adelaide, Australia. He uses a spreadsheet model to calculate collection and treatment costs for systems ranging from on-site control (all treatment-no collection) to a completely centralized system. The summary results for capital costs, annual operation and maintenance (O&M), and total costs are shown in Figures 10-5 to 10-7. All values are in 1997 Australian dollars.

The capital cost per service for treatment plants decreases rapidly from over $7,000A to a minimum of around $1,000A at a very large system serving one million customers. However, the unit treatment costs are only about $1,500A per service for 1,000 services and about $1,100A per service for 10,000 services. Thus, of the total cost savings of about $6,500 per service as treatment goes from one to one million services, $6,000A or over 90% of the total potential savings in treatment are achieved at the 1,000 service size.

Offsetting the reduction in treatment plant costs per service is the increasing collection system costs per service that range from zero to about $5,000A. Operating costs for treatment are the most significant O&M cost. They decline from about $300A per service per year for individual systems to $50A per service per year for one million services. Here again, about 80% of the savings in O&M costs can be achieved by a system with 1,000 services. The total annualized cost (amortized construction plus O&M) for this case study, shown in Figure 10-7, indicates continually decreasing unit costs for the originally assumed density. However, virtually all of the economies of scale are realized in going from 1 to 100 services. Further increases in the number of services bring only a small added gain in savings. If density decreases, then a minimum cost is reached at about 100 services. Interestingly, Clark's (1997b) conclusions are similar to the results obtained by Adams et al. (1972).

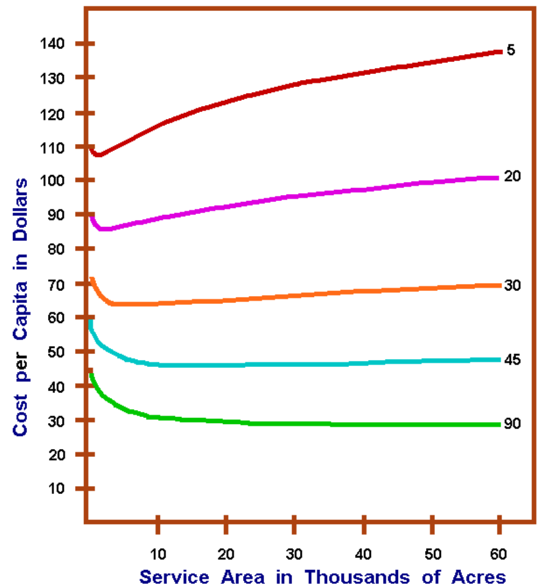


Figure 10-4. Total costs of wastewater collection and treatment systems (Adams et al. 1972). Curves represent average cost functions of collection and treatment (Numbers on curves represent population densities of number of persons/acre).

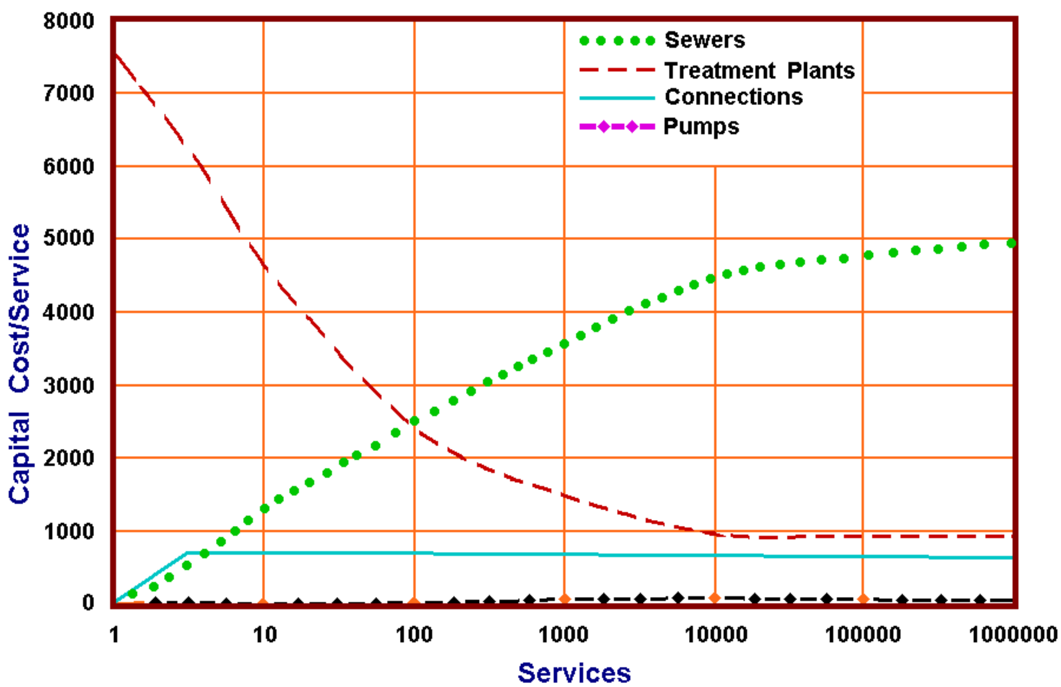


Figure 10-5. Service scale versus capital costs for components of a sewerage system. Costs are in 1997 Australian Dollars (Clark 1997b)

.

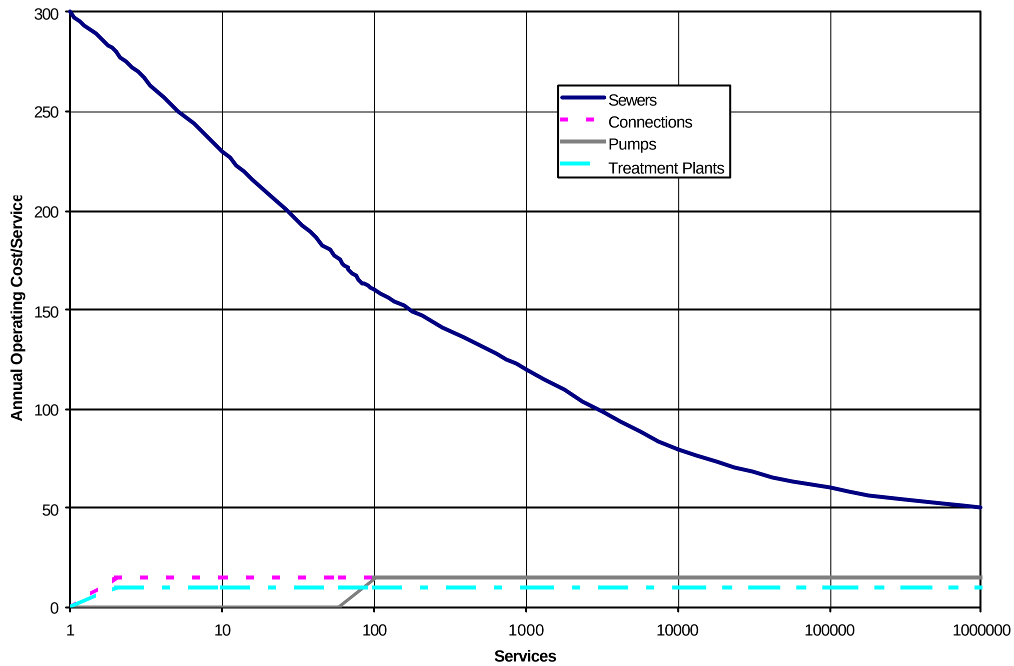


Figure 10-6. Service scale versus operating costs for components of a sewerage system. Costs are in 1997 Australian Dollars (Clark 1997b).

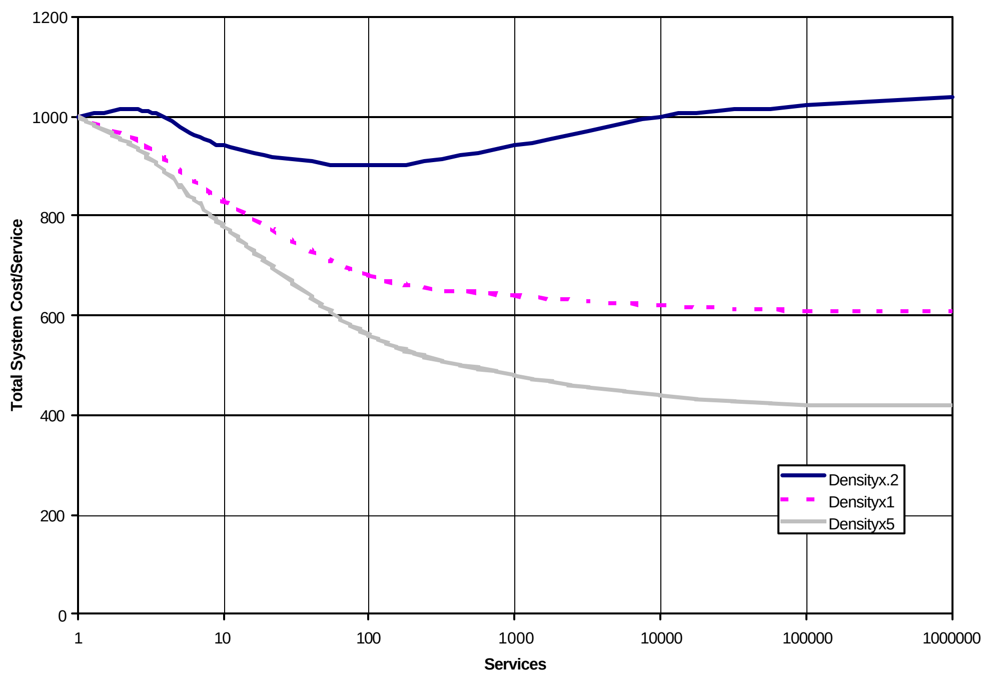


Figure 10-7. Effect of varying density of development on the minimum sewerage system cost/service and scale at which the minimum occurs. Costs are in 1997 Australian Dollars (Clark 1997b).

Costs of Infrastructure Components

Capital cost estimating equations for conveyance systems, pump stations, storage facilities, water treatment, and wastewater treatment plants are shown in Table 10-7. The general form of all of these cost equations is:

$$C = aX^{b} \qquad \text{Equation 10-1}$$

where: C=cost, and
X=size

The two parameters, a and b, are determined from fitting a power function to the available data. The traditional way to estimate a and b was by plotting the data on log-log paper and finding the parameters of the resulting straight line approximation of the data in log-log space. Now, it is simple to find a and b from a least squares regression that is built into contemporary spreadsheets.

The exponent, b, represents the economies of scale factor. If b is less than 1.0, then unit costs decrease as size increases. All of these equations shown economies of scale for the output measures of either flow or volume. Pipe flow exhibits very strong economies of scale with b <0.5. The economies of scale factor for treatment plants is about 0.7. A generic economies of scale factor that has been used for years is b = 0.6 (Peters and Timmerhaus 1980). All of the cost equations shown in Table 10-7 are updated to 1985. In order to update them to 1998 $, the resulting estimated cost should be multiplied by 1.41.

Cost of Piping

Dames and Moore (1978) reviewed the results of 455 sewer construction projects as part of a nationwide study of sewer costs. They summarize the average construction costs of sanitary sewers per foot of pipe for pipes ranging in size from six to 72 inches. These costs have been updated to 1998 values. Also, they estimate the range of design flows for each pipe diameter. The results are shown in Table 10-8. A plot of construction costs versus pipe diameter is shown in Figure 10-9. A linear relationship is apparent and this line was forced through the origin. The resulting equation is:

$$C = 14.991D \qquad \text{Equation 10-2}$$

Where: C = construction cost/foot in 1998 $,
D = pipe diameter in inches.

Simply stated, pipe construction costs per foot may be estimated as $15 multiplied by the pipe diameter in inches.

Table 10-7. Typical capital cost equations for water resources facilities[1].

Facility	Units	1985 Cost Equation Capital Cost[1]	Range	Reference	Time
A. Conveyance					
1. Force main	$/ft	$C=6.97D^{1.19}$	$6 \le D \le 72$ in.	1	Fall, 1977
2. Gravity mains	$/ft	$C=5.08D^{1.19}$	$6 \le D \le 72$ in.	1	Fall, 1977
	$/ft-mgd	$C=150Q^{.46}$	$.13 \le Q \le 43$ mgd	1	Fall, 1977
3. Open channel	$/ft-mgd	$C=12.1Q^{.41}$	$1200 \le Q \le 5800$ mgd	2	1985
4. Tunnel	$/ft	$C=4.44D^{1.14}$	$120 \le D \le 360$ in.	3	
B. Pump Station					
1. Well Pump	1000$	$C=72H^{.64}Q^{.45}$	$10 \le Q \le 2000$ gpm	4	
	1000$		$100 \le H \le 1000$ ft		
2. Water Supply	1000$	$C=13H^{.22}Q^{.44}$	$1 \le Q \le 10$ mgd	5	
			$30 \le H \le 100$ ft		
		$C=3.8H^{.37}Q^{.76}$	$10 \le Q \le 100$ mgd	5	
			$30 \le H \le 100$ ft		
3. Wastewater	1000$	$C=27HQ^{.52}$	$.1 \le Q \le 100$ mgd	6	1976
			$10 \le H \le 20$ ft		
C. Storage facilities					
1. Reservoir	1000$	$C=160V^{.4}$	$10^4 \le V \le 10^6$ AF	7	1980
2. Covered concrete tank	1000$	$C=614V^{.81}$	$1 \le V \le 10$ mg	5	1976
3. Concrete tank	1000$	$C=532V^{.61}$	$1 \le V \le 10$ mg	5	1976
3. Earthen basin	1000$	$C=42V^{.76}$	$1 \le V \le 10$ mg	5	1976
4. Clearwell					
Below ground	1000$	$C=495V^{.56}$	$.01 \le V \le 10$ mg	5	1980
Ground level	1000$	$C=275V^{.43}$	$.01 \le V \le 10$ mg	5	1980
D. Water Treatment	1000$				
1. Package treatment	1000$	$C=580Q^{.64}$	$.1 \le Q \le 1$ mgd	5	
2. Conventional treatment	1000$	$C=680Q^{.74}$	$5 \le Q \le 130$ mgd	5	
3. Direct filtration	1000$	$C=640Q^{.62}$	$1 \le Q \le 100$ mgd	5	
4. Pressure filtration	1000$	$C=402Q^{.68}$	$1 \le Q \le 20$ mgd	5	
5. Reverse Osmosis	1000$	$C=1430Q^{.68}$	$1 \le Q \le 10$ mgd	5	
6. Ion exchange	1000$	$C=370Q^{.68}$	$1 \le Q \le 10$ mgd	5	
7. Lime softening	1000$	$C=1030Q^{.68}$	$10 \le Q \le 50$ mgd	5	
8. Corrosion cont.	1000$	$C=32Q^{.67}$	$1 \le Q \le 10$ mgd	5	
9. Activated carbon	1000$	$C=809Q^{.67}$	$2 \le Q \le 110$ mgd	5	
E. Wastewater treatment					
1. Primary	1000$	$C=2980Q^{.62}$	$1 \le Q \le 100$ mgd	6	1976
2. Secondary	1000$	$C=4375Q^{.68}$	$1 \le Q \le 100$ mgd	6	
3. Tertiary	1000$	$C=11400Q^{.72}$	$1 \le Q \le 100$ mgd	6	1976

References:
1. Dames and Moore (1978)
2. US Army Corps of Engineers (1979)
3. Merkle, C. (1983)
4. Benefield, L. D. et al. (1984)
5. Gummerman, R. C. et al. (1979)
6. US EPA (1976)
7. US Army Corps of Engineers (1981)

1) To update the resulting costs to 1998, multiply by 1.41.

The next relationship, called a production function, relates the input (pipe diameter) and the output (pipe flow). The resulting curve, shown in Figure 10-9, indicates that flow increases at the 2.64 power of pipe diameter, or

$$Q = 0.0005D^{2.6451} \qquad \text{Equation 10-3}$$

Where: Q=pipe flow in cfs

Algebraically, Equation 10-3 can be solved for D and the result substituted into Equation 10-2 to find C as a function of Q. Alternatively, as was done here, a power function was fit to C as a function of Q. The result is shown in Figure 10-10 and Equation 10-4.

$$C = 217.66Q^{0.4385} \qquad \text{Equation 10-4}$$

Equation 10-4 demonstrates the strong economies of scale for pipe flow with an exponent of 0.4385. Thus, the good news is that larger sewers are more cost effective in transmitting flow. The bad news is that probably more feet of sewer pipe will be needed per service to construct a more complex pipe network.

Hassett (1995) compares the initial cost of sanitary sewers as a function of population density. His results for construction in wet and dry conditions are shown in Figures 10-11 and 10-12 respectively. Construction in wet conditions costs roughly twice the construction costs for dry conditions. Costs per dwelling unit for two DU/acre range from a high of $10,000 for wet conditions to $5,000 for dry conditions. At 10 DU/acre, costs per DU are only $2,000 (wet) or $1,000 (dry). These results appear to be a bit unrealistic. The negative exponent of nearly –1 suggests that total costs are fixed and that the costs per unit are simply total costs divided by the number of units.

Results for sanitary sewer pipe costs as a function of DU densities are shown in Table 10-9. The feet of pipe in front of the house were determined as described above. The additional amount of "larger" pipe needed per foot of local pipe is estimated as a function of population as described earlier. The unit costs of pipes were based on the 1978 Dames and Moore study updated to 1998. The results indicate the very strong influence of dwelling unit density with base costs ranging from only $1,100 per DU at 10 DU/acre to $7,000 per DU at 2 DU/acre. The effect of population is also seen to be quite significant because of the higher unit cost for larger pipes and the extra feet per DU as population increases.

Table 10-8. Sanitary sewer pipe costs and flow rates (Dames and Moore 1978).

Pipe Diameter (inches)	Average 1998 Cost ($/foot)	Flow Range (mgd)		
		Min.	Max.	Mean
6	56	0	0.08	
8	101	0.08	0.17	0.125
10	111	0.17	0.29	0.23
12	139	0.29	0.47	0.38
15	172	0.47	0.82	0.645
18	221	0.82	1.3	1.06
21	278	1.3	1.9	1.6
24	292	1.9	2.7	2.3
27	320	2.7	3.8	3.25
30	419	3.8	4.9	4.35
36	506	4.9	8	6.45
42	588	8	11.8	9.9
48	710	11.8	17	14.4
54	793	17	22.5	19.75
60	983	22.5	29.5	26
66	1,047	29.5	37.5	33.5
72	1,136	37.5	48	42.75

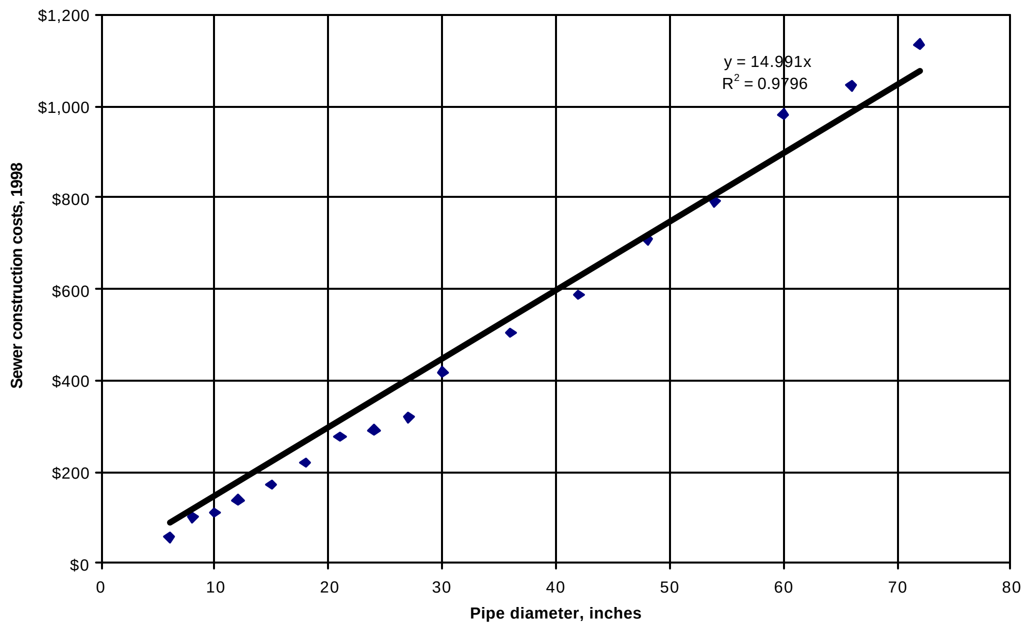


Figure 10-8. 1998 sewer construction costs per foot of length as a function of pipe diameter.

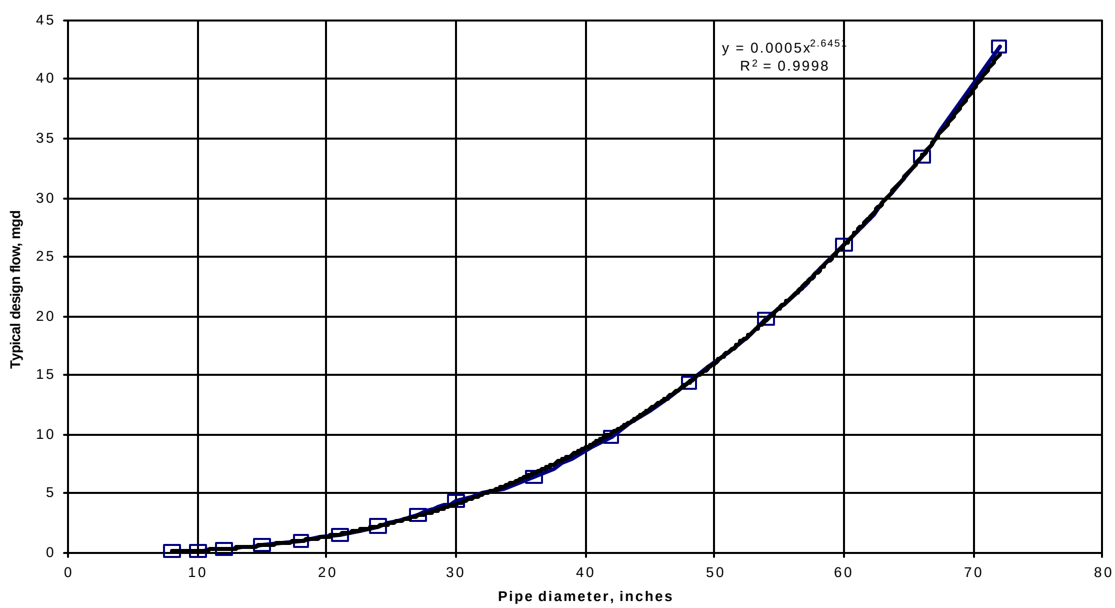


Figure 10-9. Typical flows versus pipe diameter.

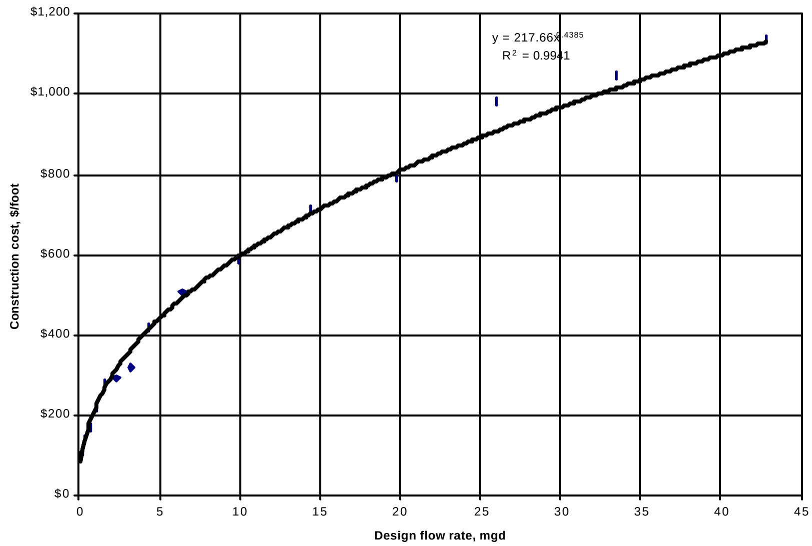


Figure 10-10. Sewer construction costs per foot of length versus design flow rate.

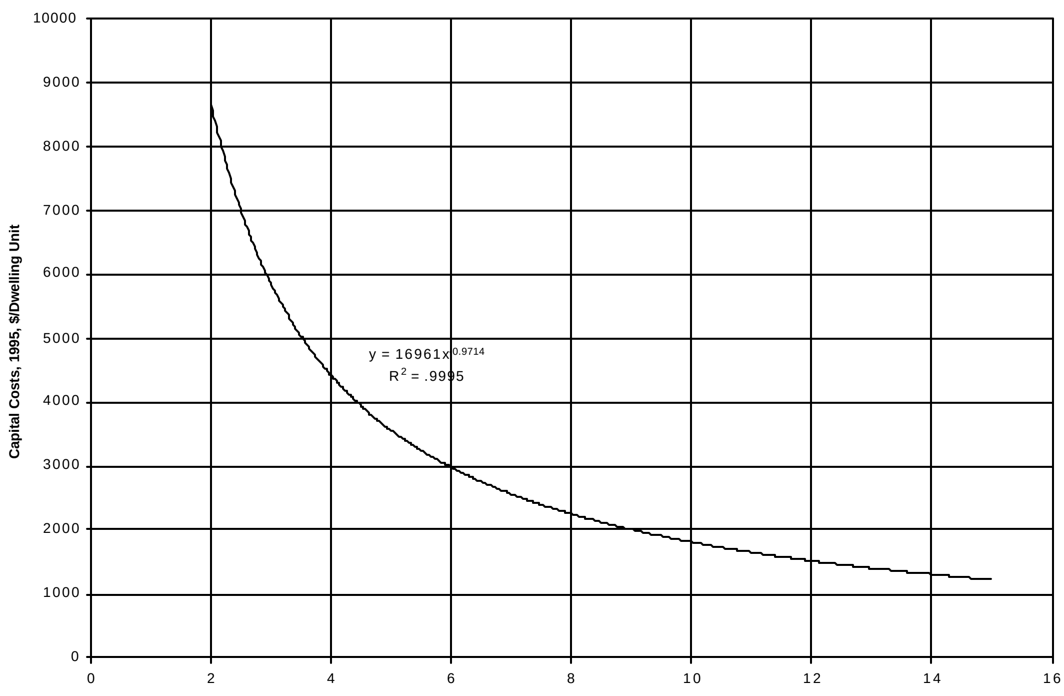


Figure 10-11. Effect of dwelling unit density on sanitary sewer construction costs in wet areas (1996).

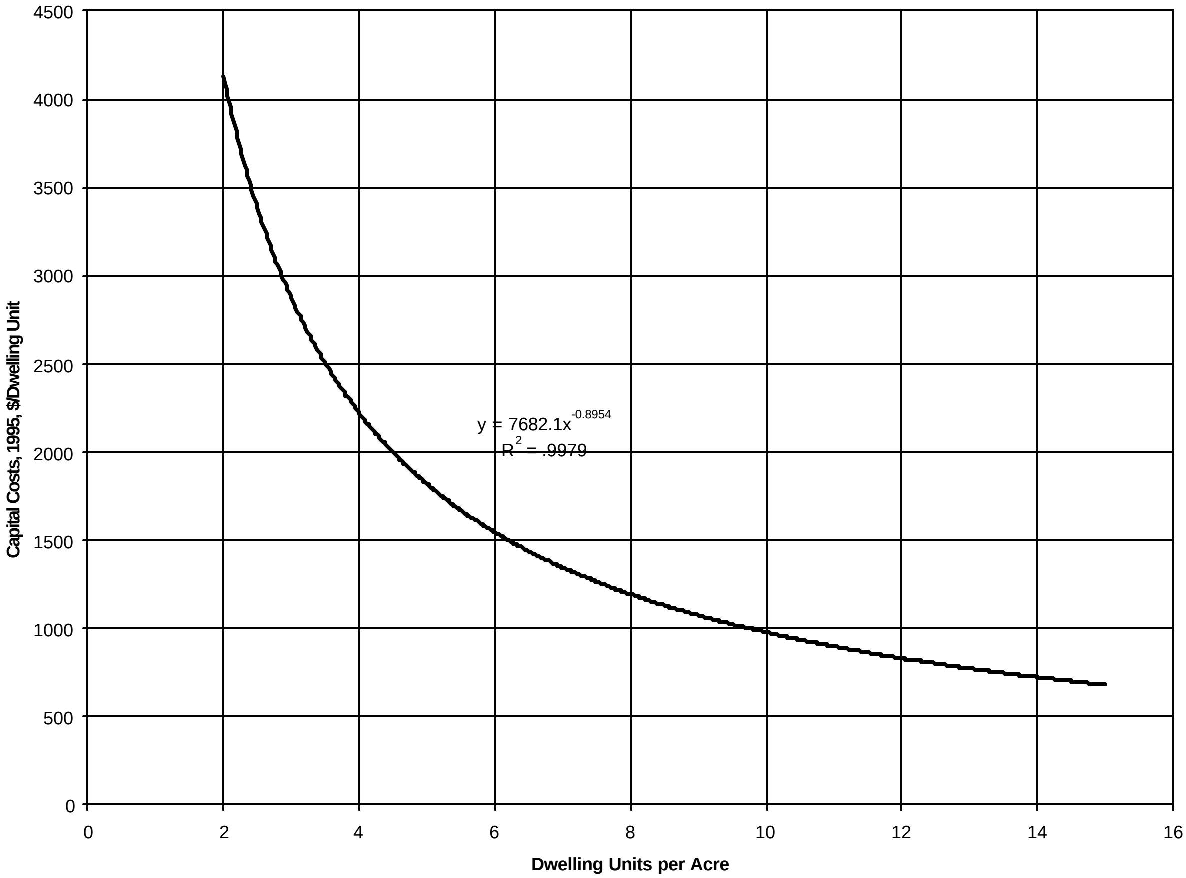


Figure 10-12. Effect of dwelling unit density on 1995 sanitary sewer construction costs in dry areas (Hassett 1995).

Table 10-9. Estimated 1998 sanitary sewer pipe costs per dwelling unit for various dwelling unit densities.

Larger/smaller Ratio:		0.15	0.2	0.4	Cost of	Total Pipe Cost for		
Dwelling Unit Density (DU/acre)	Lot Pipe (feet/DU)	Added Larger Pipe (feet/DU) for Various Population Sizes			Small Pipe[1]	Various Population Sizes[1] ($/DU)		
		1,000	10,000	100,000	$/DU	1,000	10,000	100,000
2	70	10.5	14	28	$7,000	$10,150	$11,200	$15,400
4	41	6.15	8.2	16.4	$4,100	$5,945	$6,560	$9,020
6	31	4.65	6.2	12.4	$3,100	$4,495	$4,960	$6,820
8	21	3.15	4.2	8.4	$2,100	$3,045	$3,360	$4,620
10	11	1.65	2.2	4.4	$1,100	$1,595	$1,760	$2,420

1) Assumed Unit Cost for Pipe in $/ft:

"Small Pipe"	100
"Large Pipe"	300

Cost of Treatment

The cost of treating stormwater varies widely depending on the local runoff patterns and the nature of the treatment. Cost estimates for combined sewer systems are presented in US EPA (1993) for swirl concentrators, screens, sedimentation, and disinfection. Capital costs results are shown in Figure 10-13 and Table 10-10 and operating and maintenance costs are found in Figure 10-14.

Typically, treatment will be combined with storage in order to dampen peak flows and allow bleeding water from storage to the treatment plant. This treatment-storage approach can be evaluated using continuous simulation and optimization to find the optimal mix of storage and treatment (Nix and Heaney 1988). Ambiguities in such an analysis include the important fact that treatment occurs in storage and storage occurs during treatment for some controls (e.g., sedimentation systems). As shown in Table 10-3, average stormwater flows can exceed dry weather wastewater flows for some lower DU density situations.

In order to provide a planning level estimate of stormwater treatment costs as a function of DU per acre and annual runoff, stormwater treatment is assumed to be comparable in unit cost to primary treatment. The resulting stormwater treatment unit costs in 1998 $ are shown below:

Basic primary treatment:	$0.50/1,000 gallons
Average primary treatment:	$0.75/1,000 gallons
Refined primary treatment:	$1.00/1,000 gallons

These unit treatment costs were multiplied by the estimated quantities of stormwater to get the annual cost per DU. This annual cost is then multiplied by a present worth factor of 10 to provide an estimate of the present value of this cost. The results of this cost estimate for stormwater treatment are shown in Table 10-11 that presents the estimated treatment costs per DU. These results indicate total costs per DU ranging from $129 for high density areas with relatively low runoff to $1,829 for low density developments with high runoff.

Similar analysis can be done for DWF including infiltration. A good first approximation would be to use $1.50 per 1,000 gallons for treatment cost.

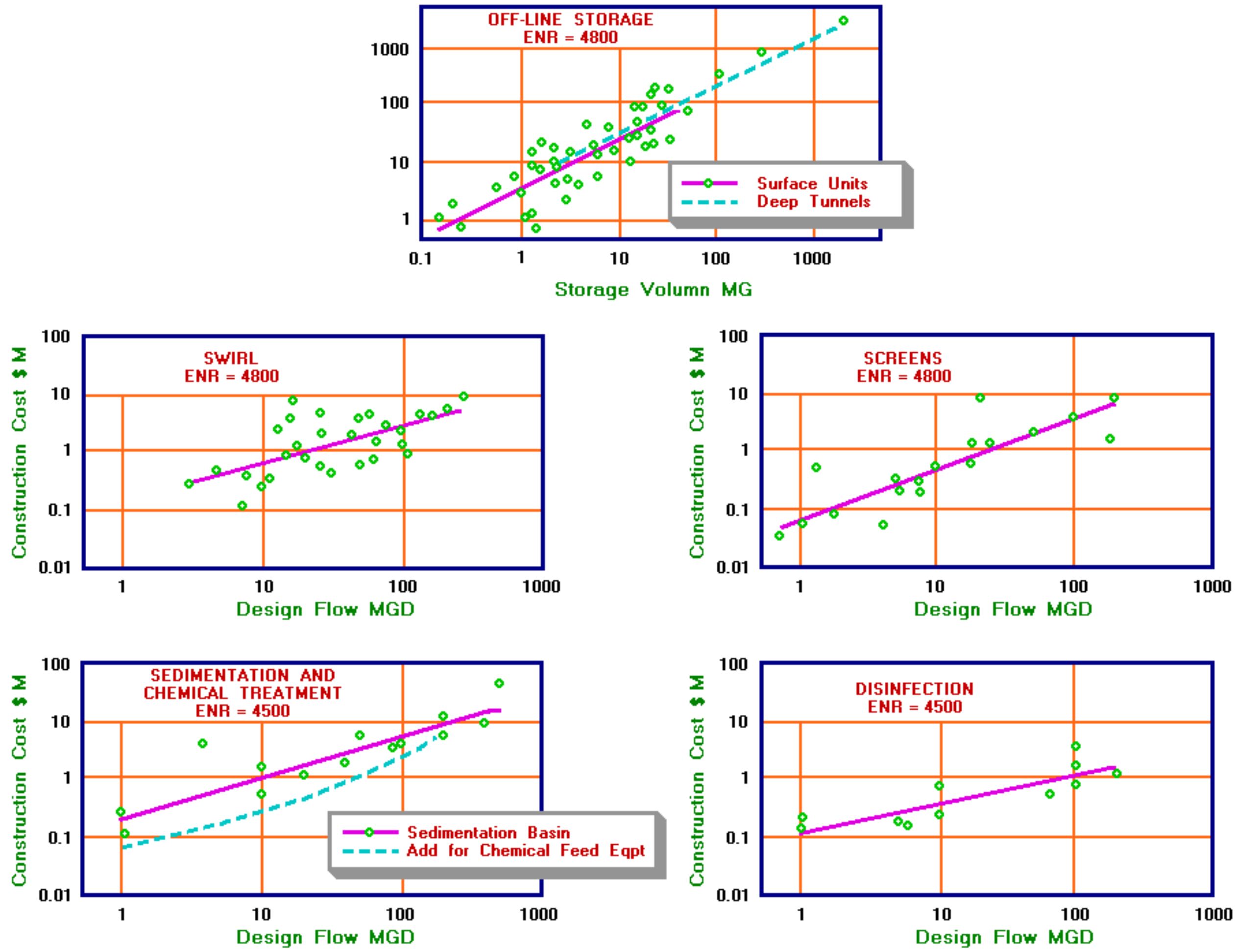


Figure 10-13. Construction costs for CSO controls (US EPA 1993).

Table 10-10. Cost equations for CSO control technology (US EPA 1993).

CSO Control Technology	**Cost Equation**	**Applicable Design Range**	**ENR Index**
Storage basins	$C = 3.637V^{.826}$	$0.15 \leq V \leq 30$ MG	4800
Deep tunnels	$C = 4.982V^{.795}$	$1.8 \leq V \leq 2{,}000$ MG	4800
Swirl concentrators	$C = 0.176V^{.611}$	$3 \leq Q \leq 300$ MGD	4800
Screens	$C = 0.072V^{..843}$	$0.8 \leq Q \leq 200$ MGD	4800
Sedimentation	$C = 0.211V^{.668}$	$1 \leq Q \leq 500$ MGD	4500
Disinfection	$C = 0.121V^{.464}$	$1 \leq Q \leq 200$ MGD	4500

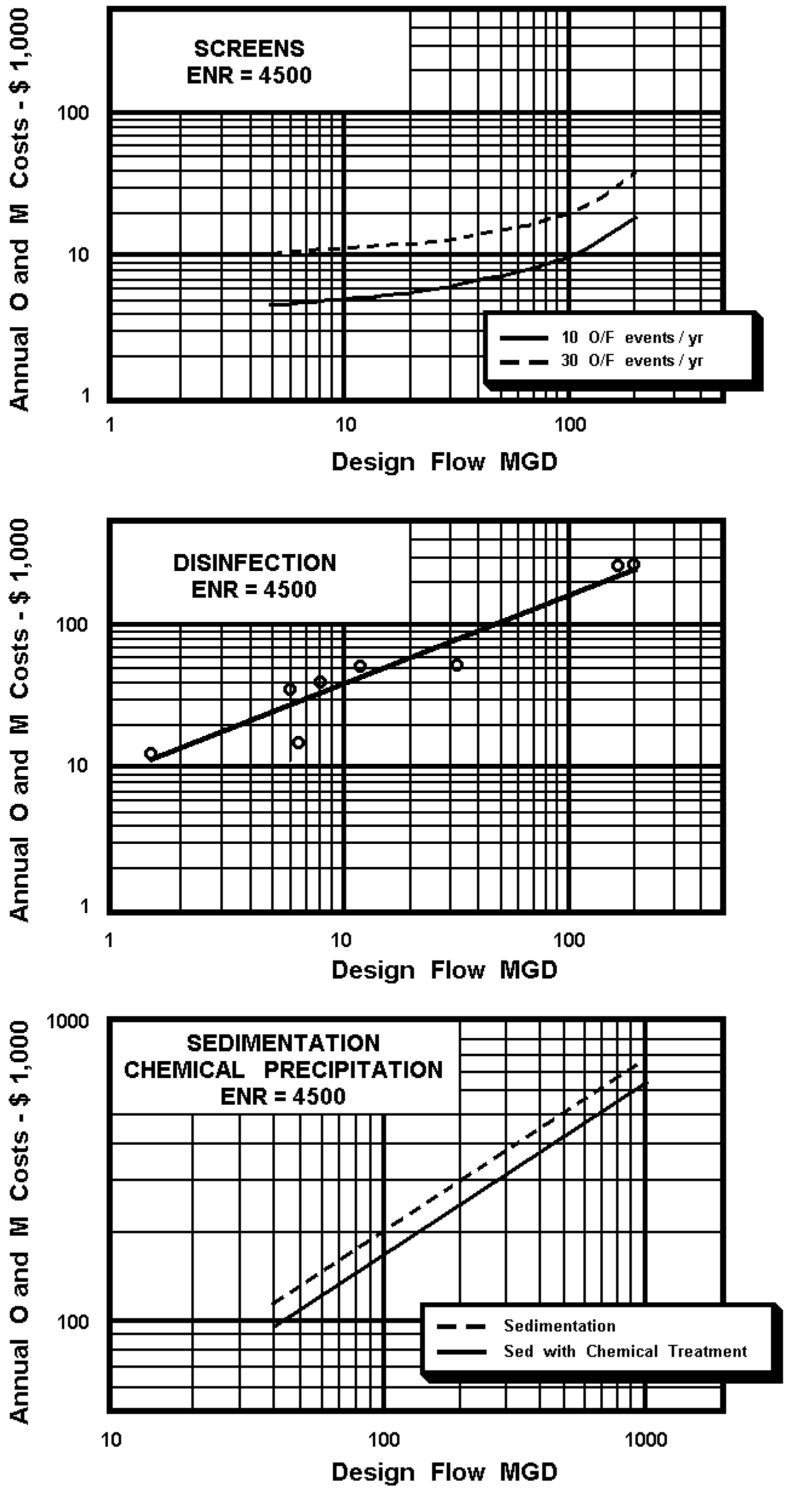


Figure 10-14. Operation and maintenance costs for CSO controls (US EPA, 1993).

Table 10-11. Present (1998) value of cost of treating stormwater runoff.

Runoff from impervious area (inches/yr.):			**10**	**20**	**30**	**40**
Dwelling Unit Density (DU/acre)	**Indoor Daily Use (gal/DU)**	**Impervious Surface (sq. ft/DU)**	**Present Value of Costs ($/DU)**			
2	180	9,780	457	914	1,372	1,829
4	180	4,690	219	439	658	877
6	180	3,760	176	352	527	703
8	180	3,445	161	322	483	644
10	180	2,756	129	258	387	515

Table 10-12. Estimated (1998) storage cost per dwelling unit[1].

Dwelling Unit Density (DU/acre)	**Impervious Surface (sq. ft/DU)**	**Present Value of Cost ($/DU)**
2	9,780	3,048
4	4,690	1,462
6	3,760	1,172
8	3,445	1,074
10	2,756	859

1) Runoff required to be stored in inches: 0.5

Cost of Storage

The total 1995 construction cost of a ground level prestressed concrete tank as a function of its volume is shown in Figure 10-15. The average unit cost ranges from $1.00/gal. for a 250,000 gallon tank to about $.25/gal. for a 10 million gallon tank.

Inspection of the cost curve indicates stronger economies of scale up to the two million gallon size. The economies of scale factor for the portion of the curve up to two million gallons in 0.51. The economies of scale factor above two million gallons is only 0.81, while the average economies of scale factor is 0.62. The estimated cost of storage for one million gallon systems using the equations in Table 10-7 indicates storage costs ranging from about $.06/gal. for earthen basins to $.90/gal. for a covered concrete storage tank.

The costs of storage reported by US EPA for CSO control projects indicate much higher unit costs as was shown in Figure 10-13. For a one million gallon facility, the unit costs range from about $4/gal. to $6/gal. in 1998 $. Recent estimates for CSO storage costs in New York City are about $9/gal. The cost of land has a major impact on the cost of storage. The reported unit costs vary from excluding land costs to valuing land at its full market value.

A preliminary estimate of the potential cost of storage per dwelling unit can be obtained using a common stormwater detention rule to store and treat the first one half inch of runoff. For the purpose of this exercise, a unit storage cost of $1.00 per gallon was used and the runoff is calculated as the runoff from the impervious area. The results are shown in Table 10-12. If on-site detention systems are used, then the cost of storage per dwelling unit ranges from $859 for 10 DU/acre to $3,048 for 2 DU/acre.

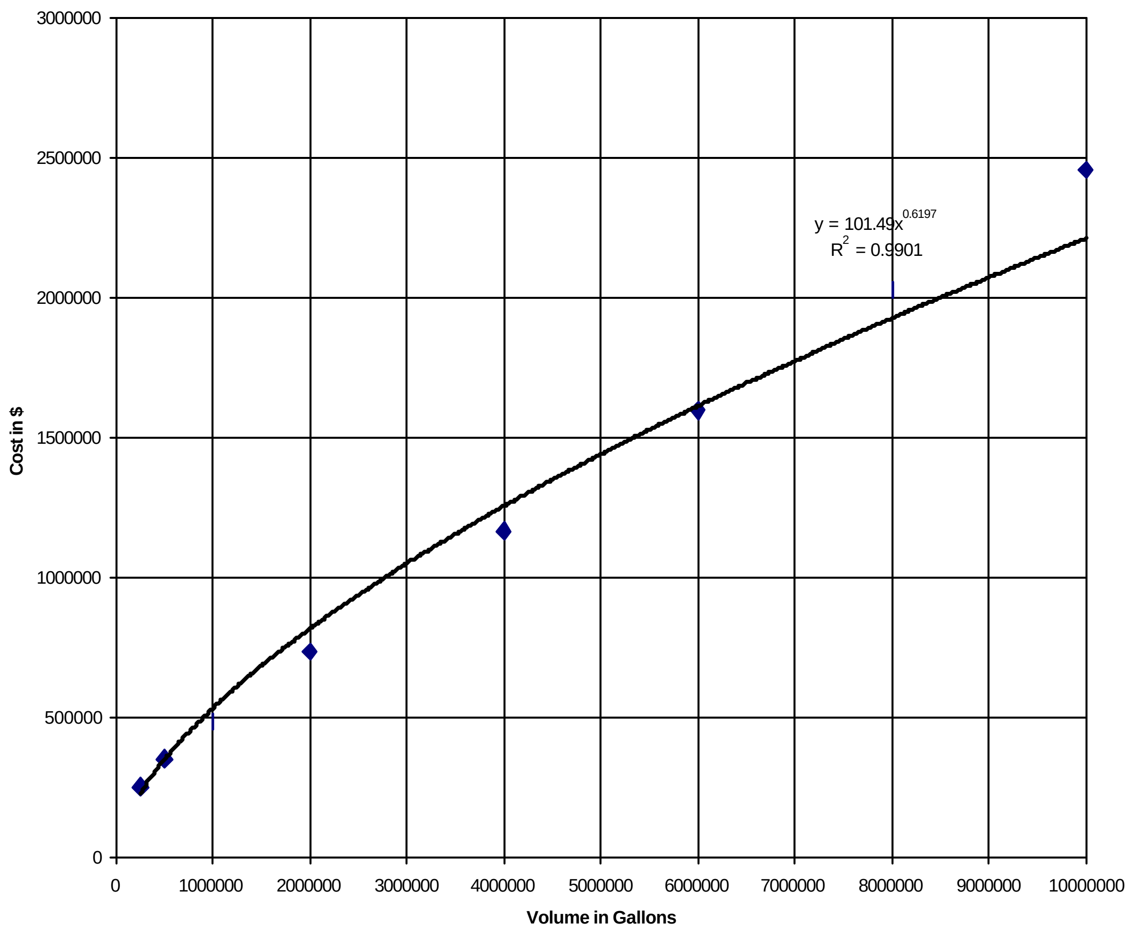


Figure 10-15. Cost of a ground level prestressed concrete storage tank in 1995 as a function of volume.

Summary of Costs for Urban Stormwater Systems

The variability in the cost per DU for urban water supply is mainly due to the amount of lawn to be watered and the need for irrigation water. In more arid parts of the U.S., most of the water entering cities is used for lawn watering. The major factor affecting the variability in wastewater treatment costs is the amount of I/I. The required lengths of pipe for water supply and wastewater systems can be approximated based on DU and ratios of the off-site pipe lengths to the on-site pipe lengths. Piping lengths per DU increase if central systems are used because of the longer collection system distances.

The costs of stormwater systems per dwelling unit vary widely as a function of the impervious area per DU and the precipitation in the area. The required stormwater pipe length per DU is about equal to sanitary sewer lengths for higher density areas in wetter climates. At the other extreme, very little use is made of storm sewers in arid areas and runoff is routed down the streets to local outlets. Also, tradeoffs exist between pipe size and the amount of storage provided. Consequently, generalizing the expected total cost of stormwater systems is difficult. The following conclusions can be reached for stormwater systems:

1. Urban sprawl has greatly increased the cost per DU for stormwater because of the large increase in impervious area per dwelling unit. Early in the 20th century, DU densities of 8-10 per acre were common. The associated impervious area per DU was about 3,000 square feet. With contemporary low density development in the range of two to four DU/acre, the square feet per DU is about 7,500. Thus, the volume of runoff per DU has increased dramatically.

2. If detention systems are needed, then storage costs per DU range from about $850 for 10 DU/acre to over $3,000 per DU for 2 DU/acre.

3. If stormwater receives primary treatment, then the costs range from $129/DU to $1,829/DU depending on runoff and DU density.

4. For wetter, higher density areas, stormwater piping costs range from $1,100/DU to $15,400/DU depending upon density and population size.

5. The development of neighborhood stormwater management systems with potential for reusing some of this water for non-potable purposes should be explored.

Financing Methods

Stable funding is an essential ingredient in developing and maintaining viable urban water organizations, whether they are stormwater utilities, watershed organizations, or some other organizational form. Integrated management offers the promise of improved economic efficiency and other benefits from combining multiple purposes and stakeholders. However, the benefits from integrated watershed management exacerbate problems of financing these more complex organizations because ways must be found to assess a “fair share” of the cost of this operation to each stakeholder (Heaney 1997). Nelson (1995) provides a current overview of utility financing in the water, wastewater, and storm water areas.

The main financing methods for urban stormwater systems are (Debo and Reese, 1995):

1. Tax funded systems
2. Service charge funded systems
3. Exactions and impact fee funded systems
4. Special assessment districts

Urban stormwater utilities have been a successful way to fund wet weather flow pollution control systems (Benson 1992, Reese 1996). Roesner, Mack, and Howard (1996) describe a wet weather flow master plan that formulates an integrated way to finance necessary stormwater infrastructure for a new development near Orlando, FL. Henkin and Mayer (1996) describe how EPA’s Environmental Financial Advisory Board (EFAB) and Environmental Financing Information Network (EFIN) can be used to create a financing strategy for implementing comprehensive conservation and management.

One of the most promising financing alternatives for wet weather flow infrastructure needs has been the development of a stormwater utility that can assess user fees (Ferris 1992, Reese 1996, and Benson 1992). A good overview of stormwater utility financing is provided in Debo and Reese (1995). Collins (1996) describes the formation of a county-wide stormwater utility in Sarasota, FL. EPA used this county as its first stormwater NPDES permit in the state.

Pasquel et al. (1996) describe the multifaceted funding mechanisms used by Prince William County, VA to fund the county’s watershed management program. The sources include a stormwater management fee based upon density and area of impervious surface, and development impact fees. The authors include a detailed discussion of the major components of the fee structure. Nelson (1995) describes alternative methods for calculating system development charges for a stormwater utility. Most systems use a combination of these methods. The following sections briefly describe the fundamentals of financing such systems.

Tax Funded System

Usually, the Public Works Department of a city is charged with maintaining and improving stormwater systems. Projects are funded through the budget of the department, whose source is mainly property tax revenue. However, if property taxes are used, then the stormwater system must compete for funds directly with public safety, schools, and other popular programs.

Service Charge Funded System

The service charge funded system uses an algorithm that divides the budget for the stormwater system by some weighting of the demand for service, (e.g., impervious areas possibly with some reduction if the area is not directly connected). This new funding method is being implemented because it has the advantage of separating the funding needs according to the function on a user pays basis. Example fees/month per acre of impervious surface from cities across the nation are shown in Figure 10-16 (Debo and Reese 1995). Debo and Reese (1995) suggest the following monthly cost ranges per residential customer for various levels of service:

1. $1.25-$2.00 for an incidental program
2. $2.50-$4.25 for a minimum level program
3. $3.33-$6.00 for a moderate level program
4. $6.00-$12.00 for an advanced level program
5. >$16.00 for an exception level program

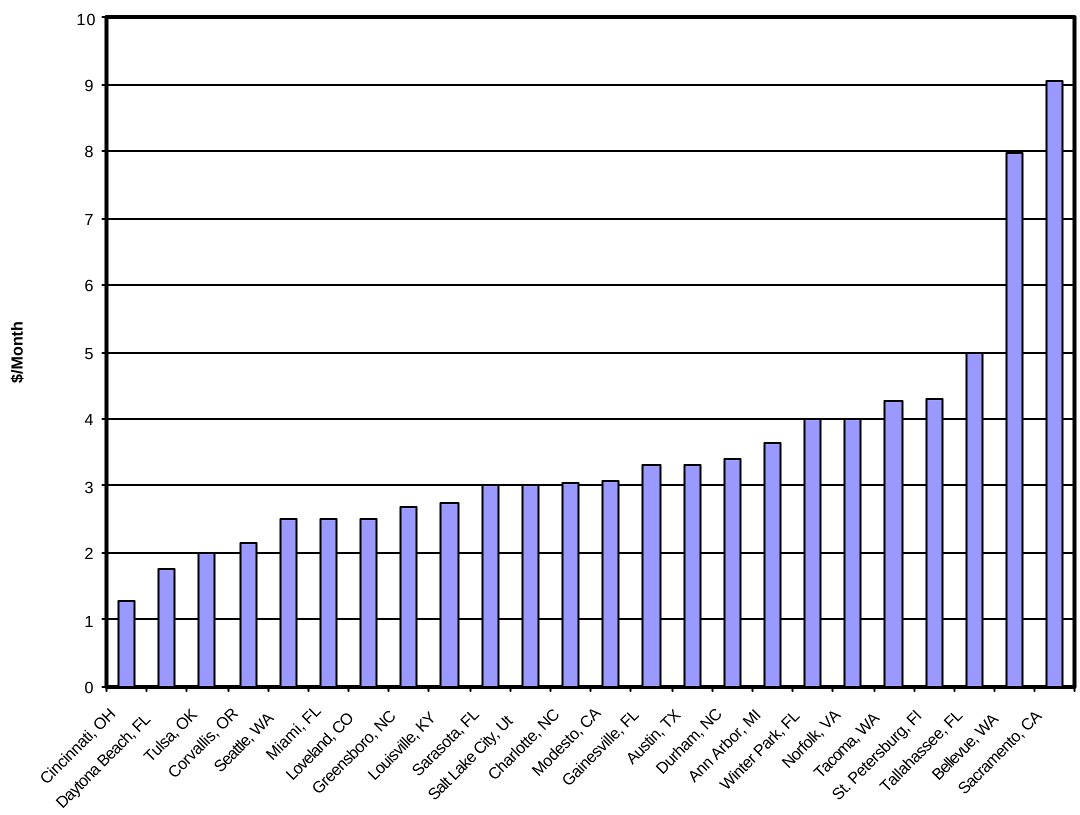


Figure 10-16. Monthly stormwater management fees (adapted from Debo and Reese 1995).

Exactions and Impact Fees

System development charges (SDC's) have emerged as the way to calculate the charges to be levied against new developments. This system charges the developer or builder an up-front fee that represents his equity buy-in to the stormwater system. Usually this fee is calculated as a measure of the depreciated value of the system, plus system-wide funding needs minus the existing users' share. The fee must be reasonable to avoid court challenges. Nelson (1995) defines the rational nexus test of reasonableness of SDCs. This tests requires:

- A connection be established between new development and the new or expanded facilities required to accommodate such development. This establishes the rational basis of public policy.
- Identification of the cost of those new or expanded facilities needed to accommodate new development.
- Appropriate apportionment of that cost to new development in relation to benefits it reasonably receives.

Care must be taken where new development results in an increase in the level of service for existing users. An important feature of this method is the ownership, or equity issue, of existing users. Usually existing users are grouped into one class for ease of calculation, however, in actuality, different groups joined at different points in time. At the time of joining, some contractual agreement (written or unwritten) was initiated. Keeping track of these agreements over time and space when setting impact fees is extremely difficult and, if not carefully done, is a key weakness of the impact fee system. Because of this added database need, and the wide variation in cost allocation methods for apportioning costs, there can be wide fluctuations in impact fee calculations. These shortcomings can be overcome, however, with better accounting and tracking of information.

Special Assessment Districts

This system funds needs within a designated geographic area by dividing the funds, usually equally, among the parcels within the area. Special assessment districts have a unique advantage in that they can follow watershed or basin boundaries. The calculation methods are inherently simple and, usually, the benefits and costs are roughly equally distributed. The disadvantage to this method is that, usually, unless a flooding disaster has occurred recently, the prospects for passage of such a district are usually very slim.

Conclusions on Finance

A variety of ways of financing stormwater management systems are available. They can enable a community to manage both the traditional flooding and drainage problem and also address issues of stormwater quality.

References

Adams, B.J., J.S. Dajani, and R.S. Gemmell (1972). On the centralization of wastewater treatment facilities. Water Resources Bulletin. 8(4), p. 669-678.

Benefield, L. D. et al. (1984). Treatment Plant Hydraulics and Environmental Engineering. Prentice-Hall.

Benson, R. B. (1992). Financing stormwater utility user fees: where are we now. Water Env. Technol. 4: 9, 59-62.

City of Boulder (1994). The 1994 Annual Utilities Report, Boulder, CO.

Clark, R. (1997a). An Exploration of the Concept, Unpublished Report #1 in the Water Sustainability in Urban Areas: An Adelaide and Regions Case Study. Department of Environment and Natural Resources. Adelaide, South Australia.

Clark, R. (1997b). Optimum Scale for Urban Water Systems, Unpublished Report #5 in the Water Sustainability in Urban Areas: An Adelaide and Regions Case Study. Department of Environment and Natural Resources. Adelaide, South Australia.

Clark, R. M. and R. M. Males (1986). Developing and applying the water supply simulation model. Journal of the American Water Works Association. 78: 8, 61-65.

Collins, P. S. (1996). Financing the future of storm water. Civil Engineering. 66: 3, 64.

Dames & Moore (1978). Construction Costs for Municipal Wastewater Conveyance Systems: 1973-1977. US EPA Technical Report. Office of Water. EPA 430/9-77-014.

Debo, T. N. and A. J. Reese (1995). Municipal Storm Water Management. Boca Raton, FL. CRC Press/Lewis Publishers.

Denver Water (1997). Comprehensive Annual Financial Report for the Year Ended December 31, 1996. Denver, CO.

Ferris, J. B. (1992). Stormwater utilities: a successful financing alternative. Multi-Objective Approaches to Floodplain Management. University of Colorado, Institute of Behavioral Science. Natural Hazards Research and Applications Information Center. 78-81.

Gilligan, C. (1996). Funding regional flood control improvements in Fort Bend County, TX. In Proc. Watershed '96 Moving Ahead Together: Tech. Conf. & Expo. WEF. Alexandria, VA.

Gummerman, R. C. et al. (1979). Estimating Water Treatment Costs. EPA-600/2-79-162a. Cincinnati, OH.

Hassett, A. (1995). Vacuum Sewers-Ready for the 21st Century. In WEF. Sewers of the Future. WEF Specialty Conference Series Proceedings. September 10-15, 1995. Houston, TX. Water Environment Federation. Alexandria, VA

Heaney, J. P. (1997). Cost allocation in water resources. Chapter 13 in Design and Operation of Civil and Environmental Engineering Systems. Revelle, C. and A. E. McGarity, (Eds). John Wiley & Sons. New York, NY.

Henkin, T., and J. Mayer (1996). Financing national estuary program comprehensive conservation and management plans: How to identify and implement alternative financing mechanisms. In Proc. Watershed '96 Moving Ahead Together: Tech. Conf. & Expo. WEF. Alexandria, VA.

Integrated Utilities Group (1995a). Rate and PIF Review/Update, Volume 1: Report, for Boulder Water, Wastewater, and Stormwater Utilities. Boulder, CO.

Integrated Utilities Group (1995b). Rate and PIF Review/Update, Volume 2: Appendixes, for Boulder Water, Wastewater, and Stormwater Utilities. Boulder, CO.

Mayer, P. (1995). Residential Water Use and Conservation Effectiveness: A Process Approach. M.S. Thesis. Dept. of Civil, Environmental, and Architectural Engineering. U.of Colorado. Boulder, CO.

Mays, L. W. and Y-K. Tung (1992). Hydrosystems Engineering and Management. McGraw-Hill. New York, NY.

Merkle, C. (1983). Cost Estimating in Water Resources. ME Report. University of Florida.

Nagle, D.G., G.W. Currey, W. Hall, and J. L. Lape (1996). Integrating the point source permitting program into a watershed management program. In Proc. Watershed '96 Moving Ahead Together: Tech. Conf. & Expo. WEF. Alexandria, VA.

Nelson, A. C. (1995). System Development Charges for Water, Wastewater, and Stormwater Facilities. Boca Raton, FL. CRC Press/Lewis Publishers.

Nix, S.J. and J.P. Heaney (1988). Optimization of storage-release strategies. Water Resources Research. 24, 11, p. 1831-1838.

Pasquel, F., R. Brawley, O. Guzman, and M. Mohan (1996). Funding mechanisms for a watershed management program. In Proc. Watershed '96 Moving Ahead Together: Tech. Conf. & Expo. WEF. Alexandria, VA.

Peters, M. and K. Timmerhaus (1980). Plant Design for Chemical Engineers. McGraw-Hill. New York, NY.

R.S. Means (1996). Heavy Construction Cost Data. 10th Annual Edition. R.S. Means Company, Inc. Kingston, MA.

Real Estate Research Corporation (1974). The Costs of Sprawl: Environmental and Economic Costs of Alternative Residential Development Patterns at the Urban Fringe. April, 1974.

Reese, A. J. (1996). Stormwater utility user fee credits. J. Water Res. Planning and Management. 122: 1, 49.

Roesner, L.A., B.W. Mack and R.M. Howard (1996). Integrated storm water planning in Orlando, FL. In Proc. of the 7th Int. Conf. Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee Urban Storm Drainage.

Schueler, T. (1995). Site planning for urban stream protection. Environmental Land Planning Series. Center for Watershed Protection. Silver Spring, MD.

Scott, K., D. C. Wang, A. E. Eralp, and D. R. Bingham (1994). Cost estimation methodologies for the 1992 CSO needs survey. In WEF A Global Perspective for Reducing CSOs: Balancing Technologies, Costs, and Water Quality, a Specialty Conference of the Water Environment Federation. WEF. Alexandria, VA.

Stadjuhar, L.E. (1997). Outdoor Residential Water Use. MS Thesis. University of Colorado, Boulder, CO.

Urban Land Institute (1989). Project Infrastructure Development Handbook. Community Builders Handbook Supplement Series.

US Army Corps of Engineers (1979). MAPS users guide and documentation. Draft Engineering Manual. EM-1110-2-XXX. Washington, DC.

US Army Corps of Engineers (1981). Unpublished data on 87 reservoirs built between 1952 and 1981.

US EPA (1976). Areawide Assessment Procedures Manual. Appendix H. EPA 600/9-76-014. Cincinnati, OH.

US EPA (1992). Manual, Wastewater Treatment/Disposal for Small Communities. US EPA Office of Water. EPA/625/R-92/005. September, 1992.

US EPA (1993). Combined Sewer Control Manual. EPA/625/R-93-007.

US EPA (1997). Response to congress on use of decentralized wastewater treatment systems. US EPA Office of Water. EPA/832/R-097/001b. April, 1997.

Whitlach, E. (1997). Siting regional environmental facilities. Chapter 14 in Design and Operation of Civil and Environmental Engineering Systems. Revelle, C., A. E. McGarity, (Eds). John Wiley & Sons. New York, NY.

Young, G. K., S. Stein, P. Cole, T. Kammer, F. Graziano, F. Bank (1995). Evaluation and Management of Highway Runoff Water Quality. Technical Report for the Federal Highway Administration.

Zuniga, A. (1997). Model for Evaluating Water Infrastructure Cost: A methodology applied to Costa Rica. Master's thesis. University of Colorado. Boulder, CO.

Chapter 11

Institutional Arrangements

Jonathan Jones, Jane Clary, and Ted Brown

Introduction

Stormwater management institutions of the 21st century must be equipped to face many challenges. Federal stormwater permitting requirements will affect most cities, even those under a population of 100,000. Funding and staffing are likely to remain tight, even though stormwater regulations and requirements continue to expand. Stormwater management will be only one of a long list of issues that must be addressed by local governments. Given the time and budget constraints typically faced by municipal staffs, they will have to decide where stormwater management lies relative to their other priorities. This is no easy task, given that the benefits of stormwater management can be elusive to quantify.

Furthermore, existing stormwater regulations are transitioning from the promulgation and implementation stages to the enforcement stage, where local governments may face legal challenges, particularly as a result of land use restrictions. Coordination among local, state, federal and private entities is and will continue to be a challenge. Stormwater management institutions will increasingly have to address both water quality and water quantity issues. In some cases, this will require retrofitting existing stormwater quantity structures to address stormwater quality issues. New stormwater management facilities will also need to be financed and constructed. Better education of the public on the significance of stormwater issues will be necessary. Research will be needed to develop new technologies for treating and retaining stormwater runoff. Institutions will have to issue guidance on complicated and often controversial issues such as riparian corridor preservation, impervious area limitations, conservation easements, innovative zoning techniques and other subjects.

Given these challenging tasks, this chapter briefly characterizes the existing models of stormwater management institutions. It then identifies five key characteristics that future stormwater management institutions will need and describes specific technical and administrative issues that these stormwater management institutions will have to address.

Existing Models of Stormwater Management Institutions

There are several existing “models” for stormwater management institutions, including watershed-based committees, local governmental agencies (such as conventional city and county public works departments and regional drainage and flood control districts), stormwater utilities, and privatized institutions. While each of these models is primarily locally based, each must function under federal and state regulations, as well as local ordinances. Any of the models could be appropriate for a given area, depending on the characteristics of the community and the watershed. Ultimately, the decision on what type of stormwater management organization is best for an area should be made by local interest groups. This may involve incorporating stormwater management concepts into an existing institution or creating a new institution (WEF and ASCE 1998). Key characteristics of the four local stormwater management institutional models are briefly highlighted below:

1. Watershed Based Committee/Institutions: "Watershed-based management is a flexible framework integrating the management of all resources--land, biological, water, infrastructure, human, economic--within a watershed" (Horner et al. 1994). These geographically-based groups of multiple public and private entities join together to pool resources and information, and develop and attain water-related goals. The primary benefits are that the institution is geographically based, can provide economies of scale and reduces the "piece-meal" approach to stormwater management. The primary drawback is the difficulty in coordinating a potentially large number of parties, establishing and determining authority, and obtaining funds. The success of watershed-based institutions is also significantly influenced by the size of the watershed (Schueler 1996).

2. Stormwater Utilities: Similar to water and wastewater utilities, municipalities assess fees/taxes to support stormwater utilities and use these funds to implement stormwater programs and facilities. The primary benefit is a steady stream of revenue dedicated to stormwater that does not have to compete with other programs and needs. The primary drawbacks are the lack of perceived need for such institutions (as compared to water and wastewater utilities) and the required creation of a new operating system that needs legal authorization to exist, operate, and assess charges (Horner et al. 1994).

3. Local Agencies: Existing local agencies, such as public works departments and urban drainage and flood control districts, can continue or expand to address stormwater issues. The primary benefit is that, in many areas, these agencies are already in place and have established authority. In addition, local governments are already responsible for land development codes and regulations with an established legal basis for reviewing and approving development plans (Horner et al. 1994). In many cases, smaller basins or subwatersheds are contained within the same political jurisdiction. These subwatersheds are more easily managed than an entire watershed, which may span multiple jurisdictions. A local government with already established authority can manage multiple subwatersheds (Schueler 1996). The primary drawbacks are limited public funding, the red-tape sometimes associated with governmental agencies, and a fragmented approach if a watershed spans several municipalities.

4. Privatization: This involves the developing, selling or partial sale of government-owned enterprises or services. Benefits of privatization include a reduction in

high “soft costs” associated with governmental organizations. Although privatization has proven to be feasible and attractive in the water treatment/distribution and wastewater treatment arenas, privatization of stormwater systems is more problematic. Privatization requires a market-driven service (Rendall 1996). Everyone in a community has the need for a water supply and wastewater treatment and everyone is willing to pay a reasonable price for these services. This is a situation that is potentially appealing to a private business. By contrast, many citizens believe that they do not benefit from expenditures on drainage and flood control systems and are unwilling to pay for such services. A private business would normally not find this to be an acceptable situation, unless the risk can be minimized. Privatization experiences in the water and wastewater arena are not necessarily transferable to the stormwater arena.

The stormwater management institution of the future may incorporate characteristics of each of these models or may look like one of these models in one area and another model elsewhere. The key to the stormwater management organization of the future is that it needs to address local issues and be structured to fit local needs. For example, in many areas, watersheds are contained in a relatively small geographic area; therefore, a watershed-based approach has a limited scope and limited number of stakeholders where coordination of stakeholders is a reasonable task. However, some watersheds, such as the Chesapeake Bay area, may cover several states making a watershed approach more difficult even though it makes the most sense physically. In some communities, environmental issues rank as a higher priority than others. In these areas, a few extra tax dollars a month toward a stormwater utility would be accepted.

The feasibility of innovative stormwater management systems is heavily dependent on trends in federal regulations. At the federal level, responsibility for urban stormwater management is spread among several agencies including the U.S. Environmental Protection Agency (USEPA) (stormwater quality), the U.S. Army Corps of Engineers (flood control and wetlands) and the Federal Emergency Management Agency (FEMA) (flood control). Better integration of these agencies could have a significant positive impact on urban water management (ASCE 1996b). An evaluation of the effect of federal regulations is beyond the scope of this chapter. However, the suggestions presented here are expected to be compatible with the existing federal regulatory framework. Similarly, state involvement in stormwater management is often fragmented between water quality control entities, water quantity entities and other regulatory programs.

Regardless of the “label” that stormwater management institutions receive, they will first need to establish a long-range strategy by defining program objectives, assessing existing conditions, and establishing a program framework. Next, they will need to select and implement a complementary set of BMPs. Finally, they will need to evaluate the program by assessing the effectiveness of these BMPs and then modify their strategy as needed (WEF and ASCE 1998). Stormwater management institutions will be required to address both technical and administrative stormwater-related issues and have the characteristics described in the remainder of this paper.

Required Characteristics of Stormwater Management Institutions

Urban stormwater management institutions for the 21^{st} century will need to incorporate five key concepts:

1. Integration: Given the probability of tight budgets and limited staffs, stormwater institutions will need to coordinate a diversified staff to address both stormwater quality and quantity issues. These personnel will also need to address related engineering, scientific, legal and planning issues. If a watershed-based model is chosen for an area, integration among various stakeholders in the watershed is necessary for the success of the watershed program.

2. Flexibility: Functioning primarily at the local level, stormwater management institutions will need to be flexible enough to meet the specific stormwater challenges of their community and/or watershed. Stormwater management cannot be approached from a "one-size fits all" perspective. Examples of flexibility include consideration of area-specific receiving water characteristics and alternative pollutant control approaches such as pollutant "trading."

3. Efficiency: These institutions must be able to function under tight budgets and limited staffs, while the institutions' responsibilities grow under increased stormwater permitting requirements. Technology such as geographic information systems (GIS), the Internet and databases, should be used where appropriate to efficiently transfer and share information between engineers, planners, scientists, citizens, and others. Successful stormwater management strategies and useful data should be shared throughout the country through publications, conferences, the Internet and other means.

4. Effectiveness: Stormwater management institutions will need to implement stormwater management practices and programs that result in both water quality protection and water quantity control. Monitoring programs should be used to assess the effectiveness of stormwater management practices. Institutions will have to demonstrate that water quality is measurably improving in order to justify stormwater-related expenditures.

5. Responsiveness: Stormwater management institutions will need to be able to respond and adapt to changes in the field. Stormwater facility design criteria must be modified periodically as new technologies become available and as design standards are refined. Local government staff will also need to work diligently to stay abreast of developments related to stormwater. Similarly, considerable effort must be devoted to staying current with computer-based technological advances.

Specific Issues to be Addressed by Stormwater Management Institutions

Financing

The ability of stormwater management institutions to adequately fund and finance stormwater-related expenditures will perhaps be the greatest challenge for these institutions, particularly when the public resists new taxes and service fees. Funding is needed to cover annual operating expenditures such as administration, maintenance and debt service. Financing is needed to pay for capital improvements. Stormwater management institutions will need to:

1. Function under decreased federal funding.

2. Coordinate with state, local and federal agencies and the private sector to allocate funding among various water quality and quantity issues.

3. Prioritize expenditures to meet the growing water-related infrastructure development and rehabilitation costs (i.e., determine the relative priorities of CSO, SSO and urban stormwater management) (Schilling 1996).

4. Set specific and limited achievable goals, given the limited financing (Schueler 1996).

5. Develop a meaningful method of cost-benefit analysis (Jones and Jones 1989). Traditional cost-benefit analysis does not normally occur for expenditures on stormwater management projects, particularly on the water quality side. The main quantifiable benefits of stormwater improvements include improved property and recreational values (ASCE 1996b).

6. Allocate resources to ensure proper maintenance of stormwater facilities.

7. Educate the public to realize that drainage improvements are the financial responsibility of those at the “top of the hill” as well as the “bottom of the hill.” The public often believes that those that are damaged by stormwater are those who should pay. Members of the public who “live at the top of the hill” often find it difficult to accept that they are partially responsible for flooding that is occurring at the “bottom of the hill,” and hence have an obligation to pay for drainage improvements.

8. Involve local funding sources. Even if non-local funding is available, motivated local water quality advocates are essential for progress in water quality improvement.

9. Develop methods for equitably assigning costs of multi-purpose/multi-group stormwater management programs (Heaney 1986).

Identification of new funding and financing mechanisms may be required, including allocating the costs of new infrastructure between public and private entities. Because the current funding and financing of many watershed-based organizations is tenuous, strategies should include sustaining these organizations, especially if they are being considered for implementing stormwater regulatory compliance. In addition to traditional tax-based and bonding approaches, the following funding and financing sources and/or a combination of these strategies should be considered:

1. Public-Private Partnerships: involves pooling and matching public and private funds. Watershed-based strategies can help to pool funds from multiple public and private entities. Public funds need to be made available for watershed-based initiatives.

2. Fee-in-lieu of: involves charging developers a fee in lieu of requiring construction of certain site-specific BMPs. This fee can be put toward construction of a more cost-effective regional facility.

3. Incentive Programs: provide adequate incentives to encourage developers to implement appropriate BMPs or enter into watershed-based groups.

4. System Development Charges: fees charged to developers when development occurs to help fund services and facilities previously constructed in anticipation of their development. In other words, these deferred fees help recover costs of capacity built into systems to accommodate expected development. SDCs are best used in conjunction with other funding methods (Debo and Reese 1995). Nelson (1995) provides detailed guidance on calculating SDCs.

5. Stormwater Utility: assesses fees/taxes to support stormwater programs and uses these funds to implement stormwater programs and facilities.

6. Privatization: has been successful in the public water supply and municipal wastewater treatment fields because there is an assured revenue stream and because both services are real and perceived necessities. A key issue is “who pays” and in what proportion.

7. Voluntary: through public education, voluntary pollution-prevention and reduction should be encouraged to help states and localities upgrade nonpoint source programs (USEPA 1996).

Staffing: Inter-Disciplinary Approach

For a water quality and quantity management program to be effective, sufficient qualified staff must be provided. In an era of shrinking funding, staffing will be a significant issue. Better communication, coordination and delegation will be required among experts and stakeholders such as aquatic biologists/ecologists, civil/water engineers, economists, attorneys, planners, representatives of environmental and citizen's groups.

Staff members must be cognizant of stormwater quantity and quality management. The subject matter has broadened to include water quality issues such as biology, sediment and wet/dry weather distinctions. The ability to rapidly transfer and share information/data through computerized systems, including the Internet, should be used to reduce redundant efforts among staff members. Institutions will probably need to increasingly "farm-out" work to private consultants, such as aquatic biologists, rather than maintain large staffs of experts. Adjustments may also need to be made as stormwater permitting impacts smaller cities that are able to maintain the staffs required to implement the permitting process.

Administrative Authority

For stormwater management institutions to be effective, they must have adequate state and local legal authority to accomplish their mission. Authority is needed to create, adopt, and enforce ordinances and regulations. Statutory authority must exist for local entities to set up dedicated funding sources, such as a local utility (Horner et al. 1994).

For areas using a watershed approach, an area-wide agency or umbrella organization having authority to require and direct actions by each member political subdivision is needed. This type of authority is not available to most individual watershed management organizations because of multiple jurisdictional involvement or lack of statutory authority. States could assist in establishing appropriate authority by passing enabling legislation and by assisting organizations seeking to address regional stormwater regulatory issues. Considerations include:

- The umbrella organization must have an independent and continuous source of funding.
- One entity must guide implementation.
- The relative authorities and responsibilities of "overlapping" jurisdictions must be determined up-front (Jones 1988).

In any event, public works officials are advised to interact regularly with their colleagues in the city/county/watershed attorney's office because there will increasingly be questions about the extent of the institutional legal authority.

Regulatory Flexibility

The USEPA is increasingly demonstrating its willingness to consider alternatives to the "one size fits all" regulatory approach on wet-weather issues. That is, the USEPA is willing to consider a case-by-case approach. More flexibility and acknowledgment of regional and local constraints should be integrated into the regulatory process. For example, many of the regulatory considerations that apply to streams in humid areas of the U. S. have no application to streams in arid or semi-arid environments (Harris et al. 1996, Stevens 1996). There is emerging recognition that standards and regulations should realistically permit flexibility to respond sensibly to varying physical, biologic and economic conditions and needs. This can be achieved by performing common sense comparisons of benefits, costs, practicality, and cost-sharing alternatives.

A decision support system is necessary to enable flexibility in regulatory administration, that is, a collection of approaches enabling water resource planners to select consistent, appropriate actions with reasonable a priori estimates of the effectiveness of the approach. New control approaches should be developed and demonstrated to enable planners to reach protection goals. USEPA (1996) suggests that there would be value in collating watershed management techniques with information such as on water quality impacts, efficiencies, total costs and sustainability from research projects and demonstrations.

In addition, innovative approaches, such as pollutant trading which has been widely applied in the air arena, have also been applied to the water arena. The USEPA is in the process of establishing a framework for watershed-based pollutant trading. This type of approach incorporates market incentives to further water quality goals and adds flexibility to stormwater regulation (WEF 1996).

Clear Regulations and Standards

Debo and Reese (1995) succinctly summarize the importance of good regulations for achieving stormwater objectives:

> ***The stormwater management structure must bring together the institutional goals, objectives, and administration and the technical solutions using models and master plans by means of regulations, policies and ordinances. When properly conceived, legal authority spans the gap between the two by pairing institutional goals or concerns with technical solutions through the use of performance oriented criteria.***

In the future, more stormwater quality regulation is likely to occur at the state and local level, with a decreased role for the USEPA. As long as there is local commitment, knowledge, and resources, water quality is best managed on a local and/or watershed-basis, with local and state officials and staff making the key decisions. This approach is consistent with the philosophy that specific characteristics of receiving waters should dictate the necessary quality of wet weather discharges. Clear regulations and standards support efficient and effective functioning of a stormwater management institution. Regulatory considerations include:

1. Local problems must be defined clearly to provide meaningful guidance and leadership to all affected interests throughout the development of enabling legislation, regulations, and design criteria.

2. Regulations should define functions and minimum performance objectives of stormwater facilities.

3. Wet weather water quality criteria should be developed that are representative of the specific receiving water—not merely generic water quality criteria/standards.

4. Stormwater quality control programs should strive to protect designated beneficial uses of receiving waters by directing controls at pollutants that impair beneficial uses; however, it must be recognized that in some cases costs may be prohibitive to obtain all beneficial uses (WEF and ASCE 1998).

5. Published design criteria for BMPs, in performance and/or specification terms, must be provided.

6. Regulations must specify minimum submittal requirements for development activities; identify construction inspection requirements and timing; provide for short- and long-term maintenance; and provide for documentation of approvals, special requirements and inspections.

7. Developers proposing construction must obtain water quality impact approvals. There will be much more emphasis on erosion and sediment control in the future, as communities recognize the significance of this problem and the generally poor state of the practice at construction sites.

8. As wet weather criteria/standards become available within the next five to ten years, the BMPs that are now being implemented may no longer be adequate. Stormwater management institutions must plan to handle this scenario.

9. Effectiveness and implementability of nonpoint source regulations should be considered.

10. If the evidence continues to accumulate that aquatic ecosystems are destined to suffer significant damage beyond a certain percentage impervious area,

communities are projected to increasingly adopt impervious area “caps,” such as the limitations that Austin, TX already has in place.

Legal Challenges

Stormwater management institutions are likely to function in an era of increasing litigation related to “wet weather” issues. However, most programs should be able to stand these tests if they are: not in violation of state legislation or municipal charters, equitable, fairly enforced in the best interest of the general public, sound from the scientific and engineering perspectives and well-documented (Debo and Reese 1995).

Nonetheless, much litigation will likely arise from land use-type issues, such as:

1. Impervious area limitations.
2. Maximum slope limitations.
3. Mandatory riparian zone setbacks.
4. Mandatory setbacks from regulatory wetlands.
5. Mandatory setbacks from sensitive environmental features, such as sinkholes in karst terrain.
6. Lot size limitations.

For example, when a local government suggests allowing no more than 20% impervious area within a given watershed to protect urban streams, objections from the development community, governmental leaders and some citizens should be expected.

Regional Solutions

Regional solutions to stormwater issues encompass both physical and administrative approaches including regional structural stormwater facilities, pollutant trading and watershed approaches. Regional approaches to stormwater management should be encouraged and enhanced through state policy and programs.

Watershed-based approaches are often regional by definition since many watersheds incorporate numerous jurisdictions. Watershed/regional approaches to stormwater management make sense from a hydrologic point of view, but are often constrained by administrative issues such as funding, lack of legal authority, and staff continuity. Regional planning entities, while not always organized around drainage basins, are logical entities to address regional stormwater concerns. However, regional planning entities need to be active and have the resources to support stormwater regulatory compliance.

Many communities have come to recognize that larger, “regional” stormwater quantity/quality control facilities are preferable to numerous, smaller, on-site facilities for reasons related to maintenance, appearance, functional effectiveness, including multipurpose use, and cost effectiveness. Unfortunately, many such communities also lack the up-front money necessary to secure optimal sites for regional facilities and to construct the facilities, even though they provide economies of scale in the long run. Stormwater management institutions must help such communities obtain these sites.

Pollutant trading is one regional solution that has been employed in the air and water realms. One critical aspect of successful pollutant trading programs is public relations, including up-front development of partnerships and consensus (Toth 1996). Pollutant trading systems, including both point and nonpoint sources, allow discharge sources to exchange pollution control obligations in order to lower the joint costs of compliance. The potential economic and environmental advantages of trading have drawn increasing broad-based support. In May 1996, the USEPA issued draft guidelines to encourage and facilitate watershed-based effluent trading. Successful trading systems require that government provide three basic conditions: the creation and definition of an allowance, a quantitative restriction on effluent discharge, and the creation and administration of a system of allowance exchange (National Institutes of Water Resources 1996).

Podar et al. (1996) summarized progress of trading programs across the nation and provides examples of such programs (Field et al. 1997). One example in Boulder, CO involves the decision to improve stream flow, restore the riparian zone and install some nonpoint source control measures rather than upgrade the municipal treatment facility to remove more ammonia. Boulder has saved up to $3.5 million in capital costs and gained improvements to the environment, including improved streambank stabilization, reduced streambank erosion, improved filtration of runoff, improved fish habitat, more continuous protected riparian zone for wildlife and increased wetland area. Pollutant trading programs such as this one should be encouraged and solutions to administrative difficulties should be shared nationally.

Interest in watershed approaches has also increased, as evidenced by over 300 papers presented at the “Watershed ‘96” conference in Baltimore (Field et al. 1997). The watershed approach is also being driven by federal natural resources management policy. One of the key motivations for watershed-based approaches is enhanced local control and improved economic efficiency. Cost-savings can be realized through coordinated monitoring efforts and cost-effective pollutant removal for the watershed as a whole. Joint efforts include the pooling of funds, expertise and capital. In many cases, the benefits of joint efforts are multiplied beyond the initial savings. For example, the benefits of effective monitoring enable better decision-making based on more accurate and complete data (Brewer and Clements 1996).

Total Risk Management

Local governments are often involved with a variety of natural hazards, such as fire, wind, landslides and earthquakes. Stormwater-related issues are just one category of the total risks facing local governments. Risk management decision-support tools should be used to optimize the use of various control strategies/technologies for stormwater management including retrofitting, upstream pollution prevention, land management, and non-structural or minimal structural approaches (USEPA 1996). For example, risk management approaches such as the Watershed Analysis Risk Management Framework (WARMF) can be used to select management approaches based on cost, effectiveness and risk of failure of various management alternatives (Chen et al. 1998). The stormwater management institution should develop acceptable levels of risk on a watershed by watershed basis.

Maintenance

Even the best stormwater management programs and facilities fail without proper maintenance. Resources must be allocated to ensure proper maintenance of stormwater facilities. When requirements to install stormwater BMPs are imposed on private parties, without the assurance that proper maintenance will be practiced, the facilities fail to function, fall into disrepair, become unsightly and are viewed as a nuisance (Zeno and Palmer 1986). The stormwater management institution should set up requirements and guidance for appropriate maintenance. Clear policy should be developed clearly specifying who is responsible for maintenance (Horner et al. 1994).

Monitoring/Evaluation

Regular monitoring and evaluation helps to determine whether the stormwater program is achieving its goals and being administered in an efficient, cost-effective manner. Procedures can include actual environmental monitoring such as water chemistry, biological communities (e.g., aquatic life), and sediment chemistry. Monitoring program objectives must be clearly identified when initiating the monitoring program (Horner et al. 1994). Stormwater monitoring should include quick and relatively inexpensive biological tests to establish the toxicity of stormwater runoff. These tests and other chemical tests (again, which are straight forward and inexpensive) will enable the determination of problematic constituents in the stormwater by local stormwater institutions.

Clear guidance should be developed and distributed on developing practical monitoring programs that represent a compromise between the number of samples suggested by thorough statistical analysis and economic and resource considerations. Performance assessment data from existing BMP databases can be used to determine the amount of data required to evaluate the performance of new BMPs. Clear monitoring guidance is not available for several biological and ecological properties of stormwater. As more data become available, the role of BMPs in minimizing or reducing the potential toxicity of stormwater runoff should be provided (WEF and ASCE 1998).

When determining pollutant removal guidelines, more emphasis should be placed on defining the hydrologic and water quality characteristics of the receiving water. Moreover, more public and private entities should have the capability to perform these baseline studies, without an inordinate amount of training and at relatively low cost. A better understanding of the receiving water characteristics would include:

- Determining a suitable design flow representative of wet weather conditions.

- Enhancing the understanding of dose/frequency/response relationships. That is, how often can the relevant organisms receive how much of a given pollutant?

Similarly, the use of ecological endpoints or “targets” should be increasingly used to define objectives for urban stormwater quality management. This type of approach is consistent with an overall, integrated watershed management approach that links studying streams, groundwater, aquatic communities and other environmental components of interest. It also includes studying municipal WWTP discharges, industrial waster sources, CSOs, sanitary sewer overflows and discharges (WEF and ASCE 1998).

A variety of new monitoring approaches are available including a “stress-response” framework, risk assessment approaches, environmental effects monitoring and other methodologies. One innovative approach uses in situ probes and biomonitors that involve putting organisms in place for brief periods of time to measure phenomena not measured by typical chemical procedures or using special in situ organisms to detect impacts. One challenge with biomonitoring-type approaches is increased difficulty with interpreting data, whereas chemical monitoring allows easier comparison to water quality standards. As monitoring systems develop over the next decade, a balance will need to be reached between chemical and biological monitoring approaches (WEF and ASCE 1998).

Finally, stormwater management institutions should regularly evaluate the effectiveness of monitoring programs and be willing to adapt to improve their effectiveness. This evaluation should include analyzing results of water quality monitoring, return on expenditures (i.e., determining if money invested is providing benefits worth the costs), and public education.

Modeling and Performance Auditing

As a follow-up to monitoring and evaluation, modeling can be used to supplement monitoring efforts with simulations that allow prediction of both discharge and receiving water quality (WEF and ASCE 1998). However, selection of appropriate models and collection of data necessary to run these models can be time-consuming and challenging in some cases. A WEF and ASCE (1998) summary indicates that models can be used to achieve the following objectives:

1. Characterize the urban runoff with regard to temporal and spatial detail, and concentration/load ranges.

2. Provide input to a receiving water quality analyses.

3. Determine effects, magnitude , locations and combinations of control options.

4. Perform frequency analysis on quality parameters (to determine return periods of concentration/loads).

5. Provide input to cost-benefit analyses.

Although models do not replace a well-planned field monitoring program, they can sometimes be used to extend and extrapolate measured data and enhance field-sampling results.

Acquisition of high-quality data needed to support modeling affect the level of effort and costs associated with the modeling effort. Two general types of data required for modeling include input parameters needed in order for the model to function and data needed for calibration and verification. Input parameters include both quantity and quality-related data. Examples of quantity-related data include rainfall information, area, imperviousness and runoff coefficient. Examples of quality-related data include constituent concentration, median value and coefficient of variation, regression relationships, and buildup/washoff parameters. Calibration and verification data may include sets of measured rainfall, runoff and quality samples with which to test the model (WEF and ASCE 1998).

Nonstructural Source Control Strategies

Management institutions will need to place more emphasis on nonstructural source controls because, in most cases, pollution prevention is more cost effective than pollution correction. Historically, stormwater programs focused on flood control and structural controls. In the future, multilevel stormwater management is needed that combines nonstructural source controls with structural treatment controls (WEF and ASCE 1998).

Examples of source controls include: public education, recycling, stenciling stormwater inlets, removing illicit discharges, pollution prevention practices for industrial and commercial sites, modifying deploying methods and substituting products for lawn/garden care and roadway chemicals, and non-toxic product substitution from materials of construction and surface coatings/preservatives exposed to rainfall runoff (USEPA 1996). Land use ordinances including cluster zoning, conservation easements, and mandatory buffer zones are additional nonstructural strategies to protect stormwater quality. Watershed-based organizations are particularly well suited to implementation of nonstructural approaches. Innovative source control practices are expected to flourish in the future in response to stormwater quality regulations just as RCRA compliance spawned many activities that have eliminated/modified chemical usage.

Retrofitting

As the shift to recognizing the significance of stormwater quality issues continues to occur, retrofitting of existing stormwater quantity structures to improve their quantity function and also serve water quality purposes will occur. For example, to improve pollutant removal, detention ponds must be changed to increase residence time, minimize short-circuiting, and provide shallow littoral zones planted with appropriate native wetland plants. Dry detention, used widely for flood control, typically provides little pollutant removal benefits because of its short detention time, bottom discharge control, and paved channels. In many locations, codes require that street curbs and gutters be used with storm sewers to eliminate runoff ponding, even for short time periods. Many localities are eliminating this requirement to promote infiltration with grassed swales, decrease runoff volume, and improve pollutant removal (Horner et al. 1994).

Walesh (1991, 1998) reviews the historic development of the use of storage for stormwater management in the U.S. as the basis for the current retrofit potential. He stresses looking at the possibility of retrofitting existing facilities for quantity-quality control before constructing new facilities. Lower cost solutions may result. Various quantity-quality case studies are provided. Walesh and Carr (1998) describe retrofitting a combined sewer system, by means of controlled on and below street storage of storm water, to cost-effectively solve basement flooding throughout an 8.6 square mile community. Construction costs for the largely implemented system are one third of the cost of traditional sewer separation.

Technology Transfer

Technological advances offer great potential to enhance stormwater management. Conducting more "real time" analysis and system operation should increasingly become more feasible (Schilling 1996, Field et al. 1997). GIS will increasingly be linked with hydrologic modeling and decision support systems, thereby facilitating master planning. For example, the USEPA recently released a package called BASINS (Better Assessment Science Integrating Point and Nonpoint Sources) that provides links to nonpoint models including HSPF and QUAL2E using ArcView (Lahlou et al. 1996). The ability to present results of engineering analyses and to depict structural improvements will be greatly enhanced through new technology, and this will be valuable for educating the public and decision makers. Hydrologic computer models should be even easier to use than they are now. Similarly, reliable stormwater quality software will likely be developed and be easy to use. Even though stormwater management is expected to continue to occur primarily at the local level, local staff should be able to take advantage of national databases with design and implementation data to determine what measures are most appropriate for their communities. For example, the USEPA and ASCE are in the process of developing a national stormwater BMP database that will provide this type of information (ASCE 1996a).

Guidance for Practices Such as Riparian Corridor Preservation and Restoration

More guidance is needed for practices such as riparian corridor preservation and restoration. The virtues of this practice are becoming increasingly recognized, but the difficulties and limitations should be discussed as well. In many urban areas, to accomplish marked improvements in water quality and aquatic life, retrofitting stormwater quality enhancements and stream habitat improvements is necessary. Retrofitting includes: "restoring degraded urban water courses to either their original condition, or to a condition that is ecologically and aesthetically satisfactory. This includes not only the prevention of unwanted erosion, scour, and sediment deposition, but also the new methods for regaining some of their aesthetic and ecological qualities and contributing to water quality enhancement, while at the same time retaining their flood carrying capacity (which is why the streams were modified in the first place) (Torno 1989)."

Riparian corridor preservation is used as an example of the types of issues that need to be considered when preparing guidance for these practices. The value of protecting, restoring, and enhancing riparian corridors along streams in urban settings has been widely recognized. However, specific guidance on how to preserve, restore, and/or enhance riparian corridors has been lacking, particularly on the institutional side. In the last few years, efforts have begun to develop this type of guidance and should continue. For example, Herson-Jones et. al. (1995) recently provided guidance on riparian buffer programs used to mitigate the impact of urban areas on nearby streams based on a national survey and literature review. They recommend a step-by-step approach including identifying program objectives, assessing site conditions, determining a program structure, establishing minimum or standard width requirements, defining exceptions of rules for increasing or decreasing the standard width and evaluating the potential water quality benefits of the buffer program. They also address issues such as plan review and inspection, long-term buffer management, maintenance, enforcement, construction, post-construction and establishment of local ordinances.

Similarly, many federal agencies recently joined together to write Stream Corridor Restoration: Principles, Process, Practices, which provides guidelines for stream restoration and is expected to be released in 1998 (Tuttle and Brady 1996). A manual focusing on the restoration of urban streams was produced in the midwest by a partnership of federal, state and local government units (Newbury et al. 1998).

In conjunction with technical issues, guidance should address socioeconomic issues such as:

1. Selecting a variable versus fixed width approach (a "variable width" approach to delineating a buffer zone makes good sense technically, but a "fixed width" approach is much easier to administer).

2. Convincing reluctant developers and other property owners of the merits of leaving riparian zones undeveloped.

3. Legal aspects of buffer zone restrictions.

4. Promoting restoration of channels (Brown et al. 1996).

Public Involvement and Education

Public involvement is imperative to foster community ownership in stormwater programs (Debo 1982, Walesh 1993, Wright 1982). The public must be better informed to recognize that stormwater runoff is just as serious a source of pollution as CSOs. The public perception must shift to recognize the need for stormwater management. Citizens must understand how everyday activities contribute to stormwater problems. Simple pamphlets inserted into utility bills, books, videos, and displays at local events have been used successfully. Special programs such as "adopt-a-stream," and "eco-neighborhoods" are proving successful in encouraging citizens to buy into programs (Horner et al. 1994).

Furthermore, to obtain consensus and support needed for implementation of stormwater management programs, watershed stakeholders must be involved in the program development. Stakeholders can include agencies, organizations, and individuals that will be affected by the program. The ideal group of stakeholders would include interested citizens, developers, environmentalists, consultants, planners, property owners and public agencies. Early and frequent stakeholder involvement is important to develop consensus in what could otherwise be a controversial process. Issues on which participation should be sought include: sharing data and mapping, setting priorities, establishing goals, developing development criteria, measuring success, and reviewing and approving stormwater programs (Schueler 1996, Walesh 1997).

An appropriate balance must be established between the need for adequate public input versus excessive public involvement, which can actually impede progress. Given the increasing knowledge and interest of stakeholders, Walesh (1997) notes that the old DAD (decide-announce-defend) approach to urban water management must give way to the more effective POP (public owns project) model.

Conclusion

Stormwater management institutions can incorporate a variety of characteristics of the existing stormwater models or a combination of these models. The organization should be locally based with adequate legal authority to create and enforce stormwater criteria and regulations. Stormwater issues should be tackled on a limited geographic scale, preferably at the subwatershed level. The stormwater utility approach is probably the most reliable method for ensuring funds dedicated to stormwater management. Although the future of privatization in the stormwater arena is not clear, market-based incentives such as pollutant "trading" in a watershed will clearly become more popular. Watershed-based organizations face a number of hurdles. Their role in educating the public regarding stormwater issues could be significant. States could assist by performing more than a permitting role with possible activities including providing guidance to and enhancing regional cooperative efforts.

The stormwater management organization will be faced with challenges such as retrofitting existing stormwater quantity structures to meet stormwater quality needs, developing guidance for riparian corridor preservation, meeting legal challenges on land use regulations, and monitoring and maintenance of stormwater structural and nonstructural BMPs. The ability to rapidly share stormwater-related information through the use of technology, such as the Internet and GIS, should help to facilitate progress in the stormwater arena. Public involvement and education will also be keys to the success of future stormwater management efforts.

References

American Society of Civil Engineers (1996a). Nationwide BMP evaluation protocols parameters to report--final lists and suggested database structure. Letter from B. Urbonas to J. Anderson. Memphis State University. September 13.

American Society of Civil Engineers (1996b). Proposal to develop guidance manual for integrated wet weather flow collection and treatment systems for newly urbanized areas. Submitted to USEPA, Region 2. March 4.

Brewer, K.A., and T. Clements (1996). Monitoring consortiums: a key tool in the watershed approach. In Proceedings Watershed '96 Moving Ahead Together Technical Conference and Exposition. June 8-12, 1996. Baltimore, MD. Water Environment Federation. 21-23.

Brown, E., J. Jones, J. Clary, and J. Kelly (1996). Riparian buffer widths at rocky mountain resorts. In Effects of Watershed Development and Management on Aquatic Ecosystems Proceedings of an Engineering Foundation Conference. Snowbird, UT. August 4-9, 1996. ed. L. A. Roesner. American Society of Civil Engineers. New York, NY. 278-294.

Chen, C. W., R. Goldstein, J. Herr, L. Ziemelis and L. Olmsted (1998). Uncertainty analysis for watershed management. In Proceedings Watershed Management: Moving from Theory to Implementation. May 3-6, 1998. Denver, CO. Water Environment Federation. 1097-1104.

Debo, T.N. (1982). Detention ordinances - solving or causing problems? In Proceedings of the Conference on Stormwater Detention Facilities Planning, Design, Operation and Maintenance. New England College. Henniker, N.H. August 2-6, 1982. ed. William DeGroot. American Society of Civil Engineers. New York, NY. 332-341.

Debo, T.N. and A.J. Reese (1995). Municipal Storm Water Management. Lewis Publishers. Boca Raton, FL.

Field, R., R. Pitt, C.Y. Fan, J. Heaney, M.K. Stinson, R.N. DeGuida, J.M. Perdek, M. Borst, and K.F. Hsu (1997). Urban wet-weather flows. 1997 Water Environment Federation Literature Review. (Draft in Progress).

Harris, T., J.F. Saunders, and W.M. Lewis (1996). Urban rivers in arid environments—unique ecosystems. In Effects of Watershed Development and Management on Aquatic Ecosystems Proceedings of an Engineering Foundation Conference. Snowbird, UT. August 4-9, 1996. ed. L. A. Roesner. American Society of Civil Engineers. New York, NY. 421-435.

Heaney, J. P. (1986). Research needs in urban stormwater pollution. Journal of Water Resources Planning and Management. 112(1).

Herson-Jones, L.M., M. Heraty, and B. Jordan (1995). Environmental Land Planning Series: Riparian Buffer Strategies for Urban Watersheds. Prepared for US Environmental Protection Agency Office of Wetlands, Oceans and Watersheds. December. Metro Washington Council of Governments. Washington, D.C.

Horner, R., J. J. Skupien, H. Livingston, and H.E. Shaver (1994). Fundamentals of Urban Runoff Management: Technical and Institutional Issues. August. Terrene Institute. Washington, D.C.

Jones, D. E. (1988). Summary of institutional issues. In Design of Urban Runoff Quality Controls. Proceedings of an Engineering Foundation Conference on Current Practice and Design Criteria for Urban Quality Control. Potosi, MO. July 10-15 1988. eds. L. Roesner, B. Urbonas, and M. Sonnen. American Society of Civil Engineers. New York, NY. 356-358.

Jones, J. E., and D. E. Jones (1989). Stormwater quality institutional considerations. In Urban Stormwater Quality Enhancement—Source Control, Retrofitting and Combined Sewer Technology. Proceedings of an Engineering Foundation Conference. Davos Platz, Switzerland. October 22-27, 1989. ed. H. C. Torno. American Society of Civil Engineers. New York, NY. 28-35.

Lahlou, M., L. Shoemaker, M. Paquette, J. Bo, S. Choudhury, R. Elmer, and F. Xia (1996). Better Assessment Science Integrating Point and Nonpoint Sources--BASINS version 1.0. User's Manual and CD. EPA Office of Water's Office of Science and Technology. Washington, D.C. EPA-823-R-96-001.

National Institutes for Water Resources (1996). Effluent allowance trading: a new approach to watershed management. Water Science Reporter. 1-16.

Nelson, A. (1995). System Development Charges for Water, Wastewater, and Stormwater Facilities. Lewis Publishers. Boca Raton, FL.

Newbury, R., M. Gaboury and C. Watson (1998). Field manual of urban stream restoration. Illinois State Water Survey and Illinois Department of Natural Resources. Champaign, IL.

Podar, M.K., R.M. Kashmanian, D.J. Brady, H.D. Herzi, and T. Tuano (1996). Market incentives: effluent trading in watersheds. In Proceedings Watershed '96 Moving Ahead Together Technical Conference and Exposition. June 8-12, 1996. Baltimore, MD. Water Environment Federation. 148.

Rendall, C.R. (1996). Privatization: a cure for our ailing infrastructure? Civil Engineering. 66 (12): 6.

Schilling, K. (1996). Wright Water Engineers Personal Communication with Kyle Schilling. U.S. Army Corps of Engineers Institute of Water Resources.

Schilling, W. (1996). Potential and limitation of real time control. Proceedings 7th International Conference on Urban Storm Drainage. Hannover, Germany. IAHR/IAWQ Joint Committee on Urban Storm Drainage. 803.

Schueler, T.R. (1996). Crafting better urban watershed protection plans. Watershed Protection Techniques. 2 (2): 329-337.

Stevens, M.A. (1996). South Platte in metropolitan Denver—a river in transformation. In Effects of Watershed Development and Management on Aquatic Ecosystems. Proceedings of an Engineering Foundation Conference. Snowbird, UT. August 4-9, 1996. ed. L. A. Roesner. American Society of Civil Engineers. New York, NY. 439-458.

Torno, H. C. (1989). Research Needs in Urban Hydrology. In Urban Stormwater Quality Enhancement—Source Control, Retrofitting and Combined Sewer Technology Proceedings of an Engineering Foundation Conference. Davos Platz, Switzerland. October 22-27, 1989. ed. H. C. Torno. American Society of Civil Engineers. New York, NY. 568-571.

Toth, C.M. (1996). Realities of pollution control planning: St. Catherine's experience. Proc. Urban Wet Weather Pollution: Controlling Sewer Overflow and Stormwater Runoff, Specialty Conference. Quebec City, PQ, Canada. Water Environment Federation. 9-1.

Tuttle, R. W. and D. Brady (1996). Interagency stream corridor restoration handbook. In Proceedings Watershed '96 Moving Ahead Together Technical Conference and Exposition. June 8-12, 1996. Baltimore, MD. Water Environment Federation. 486-487.

USEPA Office of Research and Development, National Risk Management Research Laboratory (1996). Wet weather flow research plan. Parts I and II. August 19, 1996. Peer Review Draft.

Walesh, S. G. (1991). Retrofitting storm water facilities for quantity and quality control. Proceedings of the International Conference on Urban Drainage Technologies. Dubrovnik, Yugoslavia.

Walesh, S. G. (1993). Interaction with the public and government officials in urban water planning. Hydropolis – The Role of Water in Urban Planning. Proceedings of the International UNESCO-IHP Workshop. Wageningen, The Netherlands and Emscher Region. Germany. March – April 1993.

Walesh, S. G. (1997). DAD (decide-announce-defend) is out, POP (public owns project) is in. Water Resources Education, Training and Practice: Opportunities for the Next Century. American Water Resources Association. Keystone, CO. June 1997.

Walesh, S. G. (1998). Retrofitting stormwater storage facilities. Seminar presented to the Indiana Society of Professional Land Surveyors. Valparaiso, IN. July 1998.

Walesh, S. G. and R. W. Carr (1998). Controlling stormwater close to the source: an implementation case study. American Public Works Congress. Las Vegas, NV. September 1998.

Water Environment Federation (1996). WEF comments on EPA watershed based effluent trading draft framework. September 9. Letter to Comment Clerk. Water Docket MC-4101. U.S. Environmental Protection Agency.

Water Environment Federation and American Society of Civil Engineers (1998). Urban Runoff Quality Management. WEF Manual of Practice No. 23. ASCE Manual and Report on Engineering Practice No. 87. Prepared by a Joint Task Force of the Water Environment Federation and the American Society of Civil Engineers.

Wright, K. R. (1982). Stormwater detention: acceptance and rejection issues. In Proceedings of the Conference on Stormwater Detention Facilities Planning, Design, Operation and Maintenance. New England College. Henniker, N.H.. August 2-6, 1982. ed. William DeGroot. American Society of Civil Engineers. New York, NY. 284-299.

Zeno, D.W. and C.N. Palmer (1986). Stormwater management in Orlando, FL. In Urban Runoff Quality—Impact and Quality Enhancement Technology Proceedings of an Engineering Foundation Conference. New England College. Henniker, N.H. June 23-27, 1986. eds. B. Urbonas and L. A. Roesner. American Society of Civil Engineers. New York, NY. 235-248.

Chapter 12

Summary and Conclusions

James P. Heaney, Robert Pitt, and Richard Field

Summary and Conclusions

The purpose of this project is to conduct a thorough literature review of contemporary and projected urban stormwater management practices in the U.S. and other parts of the world. Based on this review, a framework for evaluating the effectiveness of innovative stormwater management systems for the 21st century is presented. Summaries and conclusions for the individual chapters are presented below.

Chapter 2: Principles of Integrated Urban Water Management

The results of the evaluation of the nature of imperviousness in urban areas show that the quantity of urban stormwater generated per dwelling unit has increased dramatically during the 20th century due to the trend towards more automobiles which require more streets and parking, and the trend towards larger houses on larger lots. Commercial and industrial areas need much more parking per unit of office space than they did before automobiles. Modern practices dictate devoting more of the city landscape to parking than to human habitat and commercial activities.

The net result of this major shift in urban land use is low density sprawl development that generates over three times as much stormwater runoff per family than did pre-automobile land use patterns. Much of these requirements for more and wider streets and parking have been mandated in order to improve the transportation system. Ironically, unlike water infrastructure, these services are not charged directly to the users. Rather, they are subsidized by the general public including non-users.

Chapter 3: Sustainable Urban Water Management

More sustainable water systems can be achieved by promoting water conservation to reduce the amount of water that must be imported into cities. Outdoor water use is the largest source of variability in urban water use. Reuse of treated wastewater and stormwater for nonpotable uses, such as toilet flushing and irrigation, would greatly reduce urban water supply needs. Infiltration and inflow in sewers is the largest source of variability in the quantity of wastewater going to the treatment plant. I/I amounts can be reduced considerably by improved sewer design, installation, and operation and maintenance practices. Urban stormwater varies in relative importance because of climatic variability. On the average, it is of the same order of magnitude as urban wastewater but it is much more variable.

Chapter 4: Source Characterization

The relative contributions of source areas for a specific pollutant are dependent on several factors, including the characteristics of the source area and the rain energy and volume. As expected, directly connected impervious areas contribute most of the runoff and pollutants during small rains. However, as the rain depth increases, non-paved areas can become significant.

If the number of events exceeding a water quality objective are important, then the small rain events are of most concern. Stormwater runoff typically exceeds some water quality standards for practically every rain event (especially for bacteria and some heavy metals). In the upper Midwest, the median rain depth is about six mm, while in the Southeast, the median rain depth is about twice this depth. For these small rain depths and for most urban land uses, directly connected paved areas usually contribute, most of the runoff and pollutants. However, if annual mass discharges are critical (e.g. for long-term effects) then the moderate rains are more important. Rains from about 10 to 50 mm produce most of the annual runoff volume in many regions of the U.S. Runoff from both impervious and pervious areas can be very important for these rains. The largest rains (greater than 100 mm) are relatively rare and do not contribute significant amounts of runoff pollutants during normal years, but are very important for drainage and flood control design. The specific source areas that are most important and controllable for these different conditions vary widely.

Other important source area factors affecting stormwater management concern runoff pollutant characteristics for the different areas. Particle size of particulates in the runoff greatly affect many stormwater control practices, such as detention facilities and filters. If the majority of the particles can be removed from stormwater, much of the potential problem pollutants are also removed. Unfortunately, the actual particle sizes are probably much smaller than typically assumed during the design of these facilities.

Chapter 5: Receiving Water and Other Impacts

Urban receiving water may have many beneficial uses, including:

- Stormwater conveyance (flood prevention).
- Non-contact recreation (e.g., linear parks, recreation, boating).
- Biological uses (e.g., warm water fishery, biological integrity).
- Contact recreation (e.g., swimming).
- Water supply.

With full development in an urban watershed and with no stormwater controls, it is unlikely that any of these uses can be fully obtained. With less development and with the application of stormwater controls, some uses may be possible.

There are many instances of receiving water problems associated with urban stormwater reported in the literature. Receiving water problems associated with urban stormwater are highly varied. In watersheds that are lightly developed and have relatively large receiving waters, the impacts are not as obvious as in heavily developed watersheds in more arid areas.

Chapter 6: Collection Systems

By applying new technology and revisiting traditional urban water problems with a fresh outlook, advances are being made in a wide variety of sewer related areas. Integrated storm/sanitary systems may emerge in the 21st century, that is, combined sewers may be strategically designed into new urban development. Storm runoff will be reduced by source control and infiltration BMPs and the residual of small events will be transported to the WWTP. Large events will be throttled out of the integrated system, before mixing with sanitary waste, and discharged to receiving waters. This new system will have the best of both combined systems and separate systems. The advantage of the combined system has been treatment of small runoff producing events, including snowmelt. However, the disadvantage has always been the discharge of raw sewage to receiving waters during large events. With the advantage of control technology, as the sewers and/or the WWTP reach capacity, the stormwater could be stored and/or diverted directly to receiving waters without mixing with sanitary and industrial wastes. Future systems will have a high degree of built in control.

Outlying from the new urban centers, suburban type development still exists. While less dense than the city, new suburban development will contain some of the mixed land uses found in the urban center. The collection system serving this area will be far different from the city, however, because the NPS pollution is not so severe as to warrant full treatment at the WWTP. BMPs and source control innovations will reduce stormwater impacts on the receiving water. Regional detention will be used for flood control and water quality enhancement. Sanitary wastes will be transported via pressure sewers to collector gravity lines at the city's border. The use of pressure sewers will reduce suburban I/I to near zero. In addition, the new sanitary low pressure sewers will be very easy to monitor because the age-old problem of open channel flow estimation will be avoided by using pressure lines. This provides added certainty in the flow estimation and facilitates control. Technology borrowed from the water distribution field will achieve a great level of system reliability and control. In fact, the sewer will now mirror the water distribution network, essentially providing the inverse service.

Chapter 7: Assessments of Stormwater Best Management Practices Technology

Much of this chapter's discussion is based on a plethora of information that is supported by a number of local field investigations designed to test a given BMP's effectiveness at the specific site. Still needed is a national approach, similar to NURP, that would systematize the results of a large number of investigations into a coherent, well controlled program to learn about various BMP functions, physical mechanisms, biochemistry, and design parameters. Also needed is a better measure of effectiveness. The current measure in terms of percent removal has limited value.

Another need is improvement in the design robustness for various BMPs. Until that is done, expecting a specific performance from any given BMP is unrealistic. Design robustness will improve as knowledge is gained on selecting, sizing and designing each type of BMP. Urban stormwater management also has to consider the safety and welfare of the citizens living in the urban areas. Issues of efficient site drainage, control of nuisances caused by inadequate drainage, the hazards posed by large storm events and the floods they create, and costs and benefits received for the expenditure of public dollars, have to be considered. As a result, sound stormwater management has to address not only mitigating the runoff impacts of urbanization, but also the public and community needs.

Chapter 8: Stormwater Storage-Treatment-Reuse Systems
In many parts of the country, particularly humid areas, enough stormwater can be collected to satisfy average irrigation demands. If driveway areas are eliminated due to possible problems with water quality and ease of collection, the result will be a larger tank size, however, irrigation demand may still be satisfied in a majority of cases. In arid areas, particularly those with high evapotranspiration requirements, stormwater reuse may not be justified by itself. In these cases, combining storage with treated graywater may be an option worth considering. An extrapolation of this work to urban/suburban areas of the U.S. is needed.

Chapter 9: Urban Stormwater and Watershed Management: Analysis Case Study
The results of examining the behavior of Boulder Creek in Boulder, CO. each hour of calendar year 1992 provide dramatic testimony to the influence of human activities on this stream. Boulder Creek is typical of streams in urban areas because of the intense level of activities associated with manipulating water resources as part of agricultural, industrial, mining, urban and/or other interests. The following conclusions, many of which can be extrapolated elsewhere, are drawn from this analysis:

1. Given the wide variability in flows, even from hour to hour, trying to find a single "design event" to analyze the impact of urban runoff, or any other single term in the water budget, is not reasonable.

2. A continuous water budget with a small time step (i.e., hourly) is essential in order to capture the reality of stream dynamics.

3. A process oriented approach is needed to accurately characterize what is happening in complex urban stream systems. The Boulder Creek system has evolved over the past 140 years and is a complex combination of facilities and processes such as reservoirs, canals, hydropower generation, imports, exports, and instream flow releases.

4. The wide variety of stakeholders associated with Boulder Creek continue to adapt the stream system and its management in light of changing attitudes and values. The Boulder Greenways Program implemented during the past decade is a dramatic example of these changes as is the City's recently enacted instream flow improvement program.

5. Population and land use management via the open space program have had a major beneficial impact on Boulder Creek. Thus, an integrated appraisal of land and water management is essential.

6. A key point brought out by the risk analysis is the importance of including the covariance among concentration and flow and among flows. All of these covariances help reduce the impact of stormwater runoff.

7. Ultimately, real-time water management will exist in urban areas. Thus, cities will be able to deterministically manage the concentrations and the flows entering the receiving waters throughout the year.

Chapter 10: Cost Analysis and Financing of Urban Water Infrastructure

The variability in the cost per dwelling unit for urban water supply is mainly due to the amount of lawn to be watered and the need for irrigation water. In more arid parts of the U.S., most of the water entering cities is used for lawn watering. The major factor affecting the variability in wastewater treatment costs is the amount of infiltration and inflow. The required lengths of pipe for water supply and wastewater systems can be approximated based on dwelling unit density and ratios of the off-site pipe lengths to the on-site pipe lengths. Piping lengths per dwelling unit increase if central systems are used because of the longer collection system distances.

The costs of stormwater systems per dwelling unit vary widely as a function of the impervious area per dwelling unit and the precipitation in the area. Urban sprawl has greatly increased the cost per dwelling unit for stormwater because of the large increase in impervious area per dwelling unit.

If detention systems are needed, then storage costs per dwelling unit range from about $850 for 10 DU/acre to over $3,000 per DU for 2 DU/acre. If stormwater receives primary treatment, then the cost per DU range from $129 to $1,829 as a function of runoff and dwelling unit density. For wetter, higher density areas, stormwater piping costs per dwelling unit range from $1,100 to $15,400 depending upon density and population size. The development of neighborhood stormwater management systems with potential for reusing some of this water for non-potable purposes should be explored.

The main financing methods for urban stormwater systems are tax funded systems, service charge funded systems, exactions and impact fee funded systems, and special assessment districts. A variety of stormwater management financing systems are available that enable a local community to manage the traditional flooding and drainage problem, and also address issues of stormwater quality.

Chapter 11: Institutional Arrangements

Stormwater management institutions can incorporate existing stormwater models or a combination of these models. The organization should be locally based with adequate legal authority to create and enforce stormwater criteria and regulations. Stormwater issues should be tackled on a limited geographic scale, preferably at the subwatershed level.

The stormwater utility approach is probably the most reliable method for ensuring funds dedicated to stormwater management. Although the future of privatization in the stormwater arena is not clear, market-based incentives such as pollutant "trading" in a watershed will clearly become more popular. Watershed-based organizations face a number of hurdles, however their role in educating the public regarding stormwater issues and involving the public in decision making could be significant. States could assist by performing more than a permitting role. Possible activities include providing guidance to and enhancing regional cooperative efforts.

The stormwater management organization will be faced with challenges such as retrofitting existing stormwater quantity structures to meet stormwater quality needs, developing guidance for riparian corridor preservation, meeting legal challenges on land use regulations, and monitoring and maintenance of stormwater structural and nonstructural BMPs. The ability to rapidly share stormwater-related information through the use of technology, such as the Internet and GIS, should help to facilitate progress in the stormwater arena.

Appendix

Innovative Stormwater Management in New Development: Planning Case Study[1]

Brian W. Mack, Michael F. Schmidt, and Michelle Solberg

Introduction

Background

In March 1994, the City of Orlando, FL entered into a Joint Planning Agreement with Orange County which facilitated the annexation of approximately 20 square miles (11,500 acres) of primarily undeveloped land southeast of the Orlando International Airport as shown in Figure A-1. Outlined in the Growth Management Plan Southeast Annexation Study is the City's vision for the development of this area which includes providing "opportunities for economic development, protecting natural resources, and developing an integrated and efficient system of infrastructure and social service delivery." Over the next 20 years, the entire Southeast Annexation Area is expected to develop with a mixture of land uses. City planners will regulate the development of the area, with the goal of creating a compact urban growth center. The growth center will support the future development of Orlando International Airport and will contain land uses such as office, service and industrial development, with housing to support the employment generated by the airport expansion.

The stormwater element of this planning effort included the development of a Master Stormwater Management Plan (MSMP) for the annexed area. The goals of the MSMP are to provide regional flood control and water quality protection, protect existing wetlands, and site regional facilities in such a manner that they meet both the City's and private land owners' interests. Orlando will use the MSMP to guide development as it occurs.

In November 1994, the City contracted with WBQ Design and Engineering Inc. to provide engineering services for the Narcoossee Road Improvement Project. In August 1995, the City amended its contract with WBQ to include the development of an MSMP also addresses the environmental goals of the City's Southeast/Orlando International Airport Future Growth Center Plan (May 1995) for the Lake Hart Basin. The MSMP would provide stormwater management for the projected future growth in the basin as well as for the Narcoossee Road Improvement Project.

1. This is a condensed version of the Southeast Annexation Area Lake Hart Basin Master Stormwater Management Plan, City of Orlando, Florida.

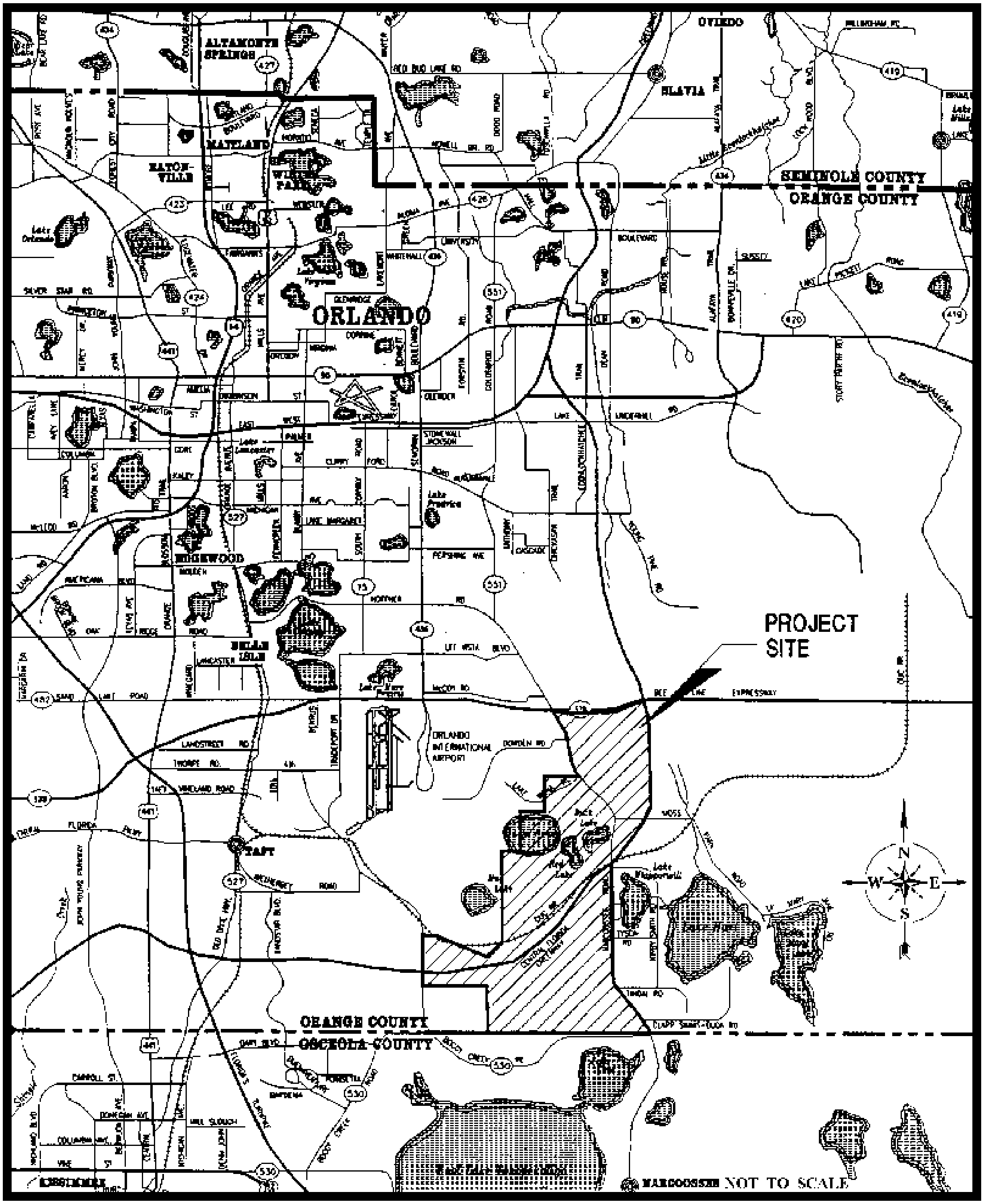


Figure A-1. Southeast annexation area vicinity map. (Reprinted courtesy of the City of Orlando, FL)

In September 1995, WBQ contracted with Camp Dresser & McKee Inc. (CDM) to provide engineering services for the development of the Lake Hart basin MSMP. The focus of this cooperative effort was to develop an MSMP with innovative options to accomplish the general goals of the City of Orlando Urban Stormwater Management Manual (OUSWMM). CDM, working with the City, outlined a “watershed based” or often called a “regional approach” to water quantity and water quality issues for this project. This included an inventory and mapping of stormwater facilities and problems and an evaluation of stormwater-related issues, alternatives, and solutions with emphasis on the management of the Primary Stormwater Management System (PSWMS) within the Lake Hart basin. The PSWMS is the major network of streams, lakes, wetlands, bridges, and culverts that convey the majority of stormwater runoff southeasterly to Lake Hart as shown in Figure A-2. This system must be operational so that the proposed secondary systems (developments) within the basin can function as designed. The MSMP will establish the framework for stormwater management within the Lake Hart.

The Master Planning Process

Stormwater runoff can be controlled by natural or man-made systems of conveyance and storage, guided development (land use controls), and the conservation of natural systems. In urban, built-out conditions, a combination of all three methods of control is necessary along with a proactive maintenance program to reach the stormwater management goals of a community. In less urban, or rural areas, stormwater management can be accomplished through land use controls and natural systems, although some conveyance and storage facilities may be needed. To gauge how well goals are achieved, levels of service (LOS) are established to quantify system performance.

The control of runoff is, therefore, a mixture of storage and conveyance engineering, land use controls, and ecosystems management. The three areas of runoff control are not mutually exclusive nor distinct. For example, land use controls affect storage and conveyance as well as natural systems. The interdependent development of conveyance and storage engineering, maintenance programs, and possibly land use controls can be of benefit to the City for planning of capital improvement programs.

Program Goals

The general goals of the Lake Hart MSMP are the development of an integrated stormwater, wetland, and open space management system that would balance preservation of natural systems with land development. The general goals are to be accomplished by meeting the following three key objectives in a cost-effective manner: flood control, pollution control, and ecosystem management (which includes wetlands protection, aquifer recharge, and water conservation). A summary of each of these

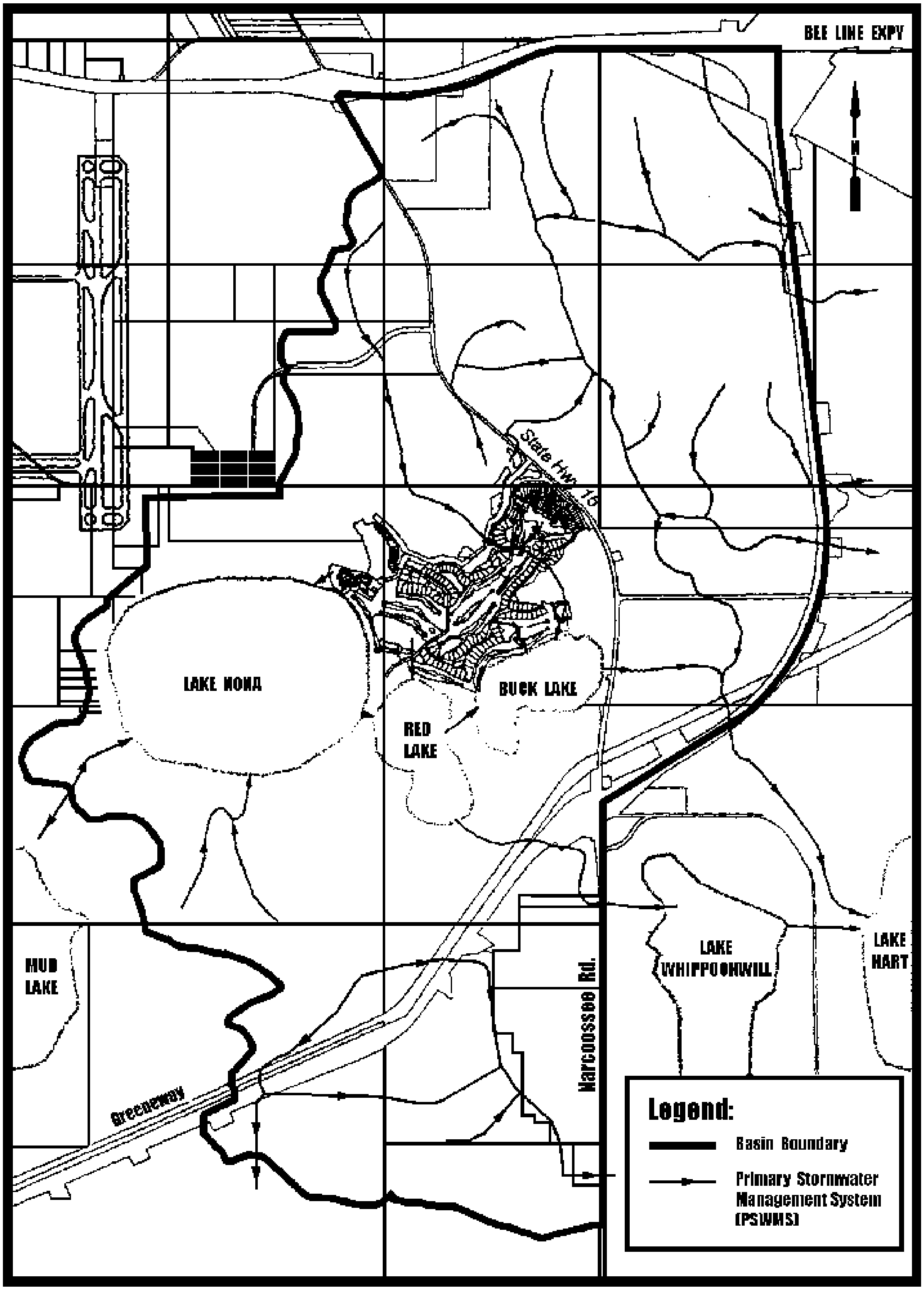


Figure A-2. Study area and primary stormwater management system. (Reprinted courtesy of the City of Orlando, FL)

objectives is presented here and further details on how goals and objectives will be met are contained in subsequent sections.

Flood Control

The flood control objective for the Lake Hart basin is locating regional facilities that will provide proper storage and conveyance of peak flows and volumes as development occurs. The facilities are to be located and conceptually designed to meet both the City's and private landowners' interests to the extent practicable (e.g., aesthetics, cost, ease of operation and maintenance). This requires close coordination with both the public and private sectors.

Water Quality Control

The water quality control objective is to provide a regional system that will treat the "first-flush" of runoff or reduce pollutant loads to the maximum extent practicable. Because of the high groundwater table and the need for fill, a wet detention system combined with pretreatment Best Management Practices (BMPs) for stormwater runoff are considered to be the most cost-effective way to meet this objective.

Ecosystem Management

The objective of ecosystem management is to develop a regional system that will protect healthy/pristine wetlands (abundant throughout the Lake Hart basin) and provide potential landscape irrigation with surface water (pretreatment and reuse).

To implement a plan that will meet these objectives, the City requested that the Lake Hart basin MSMP establish a framework for the design and review of proposed stormwater management systems within the SEAA that could be beneficially used by both City staff and developers. In general, the City wanted to supplement the stormwater management requirements of the OUSWMM with innovative technology that would address stormwater management in areas with extensively interconnected wetlands and lakes and in areas that have a high seasonal groundwater table (low infiltration potential). This framework would eventually be refined into a document similar to the OUSWMM that would eventually become the Southeast Annexation Area Stormwater Management Manual.

The City stressed the importance of training its staff to use the regional stormwater management model developed for the PSWMS in the Lake Hart MSMP. The City will use the stormwater model as a management tool to address regional stormwater related issues which may include identifying and mitigating flooding impacts from proposed land use changes as well as identifying the necessary phasing of proposed regional facilities (dependent on development schedules and conceptual plan approvals). To maintain the effectiveness of the stormwater model, City personnel will need to perform periodic updates as appropriate.

This appendix documents the MSMP strategy developed for the Lake Hart basin that can be implemented to control potential impacts to the natural stormwater system resulting from man's activities. The strategy includes a combination of land development regulations, capital improvement projects, and shared private and public partnerships (integrated resource planning) as needed to achieve the desired LOS for flood protection and water quality protection. The plan also discusses the phasing of recommended improvements to help the City implement proposed regulations and capital improvement projects in a cost-effective and timely manner.

Levels of Service

Proper LOS decisions are an essential component of the Lake Hart basin MSMP. While LOS includes retrofit, the decisions are primarily for new development. The LOS decisions will directly affect the size and cost of regional facilities and structures in the PSWMS. The OUSWMM defines primary conveyance facilities as “systems designated as outfalls from, or connections between, natural lakes and artificial regional detention facilities.” For the purposes of this case study, the primary conveyance facilities are the PSWMS.

After discussions with City staff, the LOS criteria presented in OUSWMM were amended to more clearly define existing problem areas in the Lake Hart basin. Figure A-3 illustrates the four LOS criteria considered for this study. They were formulated to protect or enhance public safety. For example, Class D provides for flood protection of first-floor elevations (FFE), while Class B provides control of flood waters so that one-half of the road is not flooded (arterial road crowns). Table A-1 lists water quantity LOS goals used to define potential problem areas (retrofit needs) in the Lake Hart basin MSMP.

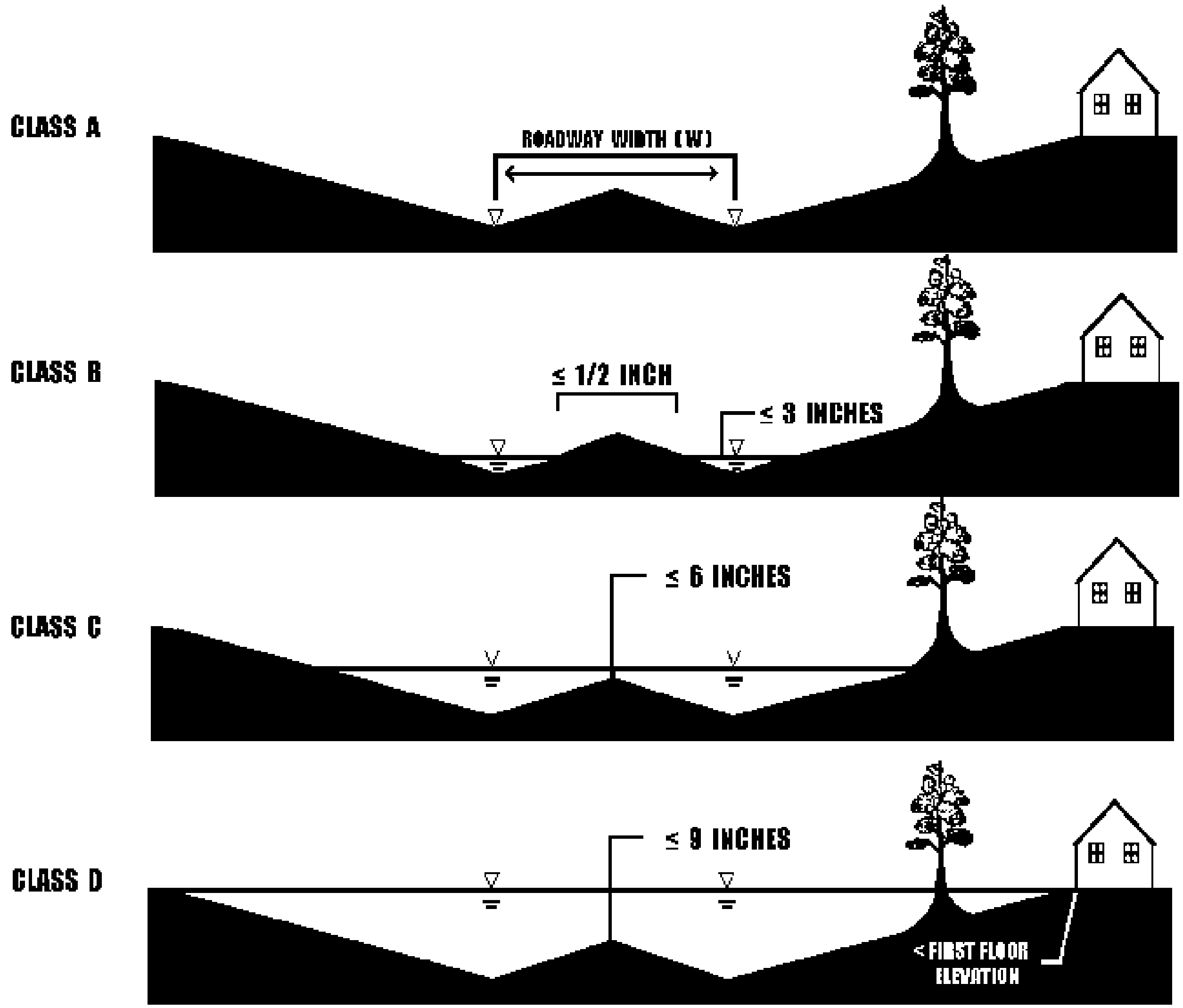


Figure A-3. Water quantity levels of service. (Reprinted courtesy of the City of Orlando, FL)

Table A-1. Existing Levels of Service For Water Quantity[1]

	10-Year		25-Year		100-Year	
Structure/Facility	10-Year	Class	25-Year	Class	100-Year	Class
Houses/Buildings	<FFE[5]	D	<FFE	D	<FFE	D
Arterial Roads [2]	½ W[6]	A	½ W	B	½ W	B
Collector Roads [3]	½ W	B	½ W	B	½ W	B
Minor Roads [4]	<0.5 ft	C	<0.75 ft	D	<1 ft	NA

Notes:
1 All storm durations are 24 hours, except the 100-year, which is 72 hours.
2 Arterial streets and highways are those which are used primarily for fast or heavy traffic.
3 Roads which carry traffic and minor streets to the major system of arterial streets and highways, including the principal entrance streets of a residential development and streets for circulation within such a development.
4 Roads which are used primarily for access to the abutting properties.
5 FFE = First Floor Elevation
6 W = Width of Road

For new development, the design criteria that are outlined in OUSWMM or this MSMP must be met. A summary of select key design criteria for primary conveyance facilities is given below:

- The design storm for new primary conveyance facilities is a 25-year/24-hour storm event. In addition, a determination of the flood stage resulting from a 100-year/three-day storm event will be made as a check of the system.
- The systems shall be designed so that existing and proposed building floor elevations shall be above the 100-year flood elevation, as determined by analyzing the 100-year/three-day event and designed to protect existing roadways from inundation during the 25-year/24-hour storm.

Note that the water quantity design criteria for new roads/development are, in some cases, greater than the LOS used for problem area identification.

Methodology

Stormwater Modeling

The primary aspect of this Lake Hart basin (MSMP) is the proper evaluation of water quantity (flooding) and water quality. A good understanding of water quantity helps determine the most effective methods of controlling flooding and protecting public safety. A proper understanding of water quality and its control is essential to ensuring the high quality of environmental protection desired by the City. Recent versions of the RUNOFF and EXTRAN blocks of the United States Environmental Protection Agency Stormwater Management Model (EPA-SWMM, Version 4.3) for water quantity were used because these models best meet the requirements of the program. The models have been verified in stormwater master plan uses throughout Florida.

The hydrologic model, RUNOFF, simulates rainfall, runoff, and infiltration characteristics of an area. It also performs simple hydrologic routing in channels, pipes, and lakes where gradients are known. RUNOFF output is electronically delivered to EXTRAN, which is a hydraulic routing model. EXTRAN provides dynamic flood routing in channels, lakes, and control structures such as bridges, culverts, and weirs. EXTRAN accounts for conservation of mass, energy, and momentum thereby predicting looping, flow reversals, and similar phenomena should they occur.

The water quality modeling framework involves identification of the water quality problems addressed by the modeling study, the structure of the model software, and the assumptions and guidelines used with the model to represent the Lake Hart basin. The Watershed Management Model (WMM) was used for the water quality analysis because this model provides evaluations consistent with EPA, NPDES and SFWMD permit requirements.

Hydrologic Model

The RUNOFF block of the EPA SWMM, which was originally developed by CDM, simulates the rates of runoff developed from subbasins using a kinematic wave approximation. Hydrologic routing techniques are then used to route the overland flows through the pipe, culvert, and channel as required. Program results can be saved for input to the EXTRAN block of Stormwater Management Model (SWMM) to perform hydraulic routing in downstream reaches. A more complete documentation of the model's background and theory can be found in the SWMM 4.3 user's manual.

Hydraulic Model

SWMM EXTRAN is a hydraulic flow routing model for open channel and/or closed conduit systems. It uses a link-node (conduit-junction) representation of the stormwater management system in an explicit finite difference solution of the equations of gradually varied, unsteady flow. EXTRAN receives hydrograph input at specific junctions by file transfer from a hydrologic model, such as RUNOFF or TR20, and/or by manual input. The model performs dynamic routing of stormwater flows through the PSWMS to the points of discharge or outfalls. Since it is dynamic, it simultaneously considers both the storage and conveyance aspect of stormwater management facilities. The program will simulate branched or looped networks; backwater due to tidal or nontidal conditions; free-surface flow; pressure flow or surcharge; flow reversals; flow transfer by weirs, orifices, and pumping facilities; and storage at online or off-line facilities. Types of conduits that can be simulated include circular, rectangular, horseshoe, elliptical, and basket handle pipes, plus trapezoidal or irregular channel cross sections. Simulation output takes the form of water surface elevations and inundated areas at each junction and flows and velocities at each conduit. The SWMM 4.3 user's manual includes further details.

Water Quality Model

WMM is a screening level water quality model used to develop relative projections of long-term pollutant loadings on an annual basis. Relative comparisons of land use and BMP implementation impacts on pollutant loads can be made. Application of the screening level model incorporates detailed data collected for each hydrologic unit used in the water quality model SWMM. WMM was applied to provide a relative evaluation of nonpoint source pollution management strategies that address water quality problems over long-term periods. WMM is a spreadsheet model for estimating annual nonpoint source loads from direct runoff based upon land use specific event mean concentrations and runoff volumes. Data required to use the nonpoint source model include event mean concentrations (EMCs) for each pollutant type, land use, average annual precipitation, annual baseflow, and average baseflow concentrations. A detailed discussion of the methodology applied in WMM can be found in the CDM WMM users manual (CDM, 1992).

The WMM model does not consider the potential in-lake or in-stream chemical, biological, or physical modification of the pollutants, nor is it intended for this purpose. WMM estimates the total load from runoff (and baseflow) to receiving waters and, as such, represents the worst case (i.e., the loading without improvement or assimilation in the receiving waters). As a next step, ecological management planning can define biological water quality levels of service so that in critical areas, more detailed, in-lake and in-stream water quality modeling can be completed to augment the Lake Hart MSMP results.

For the Lake Hart basin MSMP, WMM was used to generate estimates of average annual pollutant loadings for existing and future conditions based upon local rainfall statistics. The model relies upon EMC factors for different land use categories to calculate pollution loadings. Because the model is spreadsheet based, it can be easily applied to screen the pollutant loading reductions that can be achieved by various BMP alternatives. A series of different BMP alternatives can be screened to identify BMP requirements that will adequately mitigate existing and projected long-term water quality problems within the watershed.

Hydrologic Parameters

Hydrologic model parameters used for the model simulations are described below.

Subbasin and Hydrologic Unit Areas

For modeling purposes, the Lake Hart basin was subdivided into 51 subbasins for which land use, soil, and topographic characteristics were compiled. Subbasin area averaged approximately 150 acres with a minimum of 17 acres and a maximum of 1300 acres. For the alternative evaluations, these subbasins were further partitioned into 103 hydrologic units to account for the proposed regional facilities.

Rainfall Intensities and Quantities

There are three rainfall stations within the vicinity of the Lake Hart study area. The Boggy Creek rain gauge and the Lake Hart rain gauge are maintained and operated by Orange County, FL. The third rain gauge is the Orlando-McCoy Airport (Orlando International Airport) Station Number 6628 and 6638, and is monitored by the U.S. Department of Commerce, National Climatic Data Center. The Boggy Creek rain gauge is approximately one mile to the west of the study area and has been recording rainfall data at five minute intervals since August 1987. The Lake Hart rain gauge is approximately one mile to the southeast of the study area (within the same basin) and has been in existence since March 1995. The station at the Orlando International Airport station is one mile east of the study area and records rainfall data in 15 minute intervals. The average annual rainfall for the 1942 to 1993 period of record is 49.7 inches. The general locations of these rain gauges are shown on Figure A-4.

Rainfall For Water Quality Modeling

Wet and dry season rainfall quantities for determining nonpoint source pollutant loading projections were also determined. The rainfall volume for the wet season, which occurs from June through September, is approximately 28.1 inches. The rainfall volume for the dry season, which occurs from October through May, is approximately 21.6 inches.

Rainfall for Runoff Modeling

Design rainfall data for the Lake Hart MSMP were obtained from the OUSWMM and the South Florida Water Management District in the form of rainfall quantities and distributions (30-minute intervals) for each design storm (2-, 10-, 25-year, 24-hour, and the 100-year, 72-hour). Rainfall quantities are:

- 100-Year/72-Hour - 14.4 inches of rainfall
- 25-Year/24-Hour - 8.6 inches of rainfall
- 10-Year/24-Hour - 7.4 inches of rainfall
- 2-Year/24-Hour - 4.8 inches of rainfall

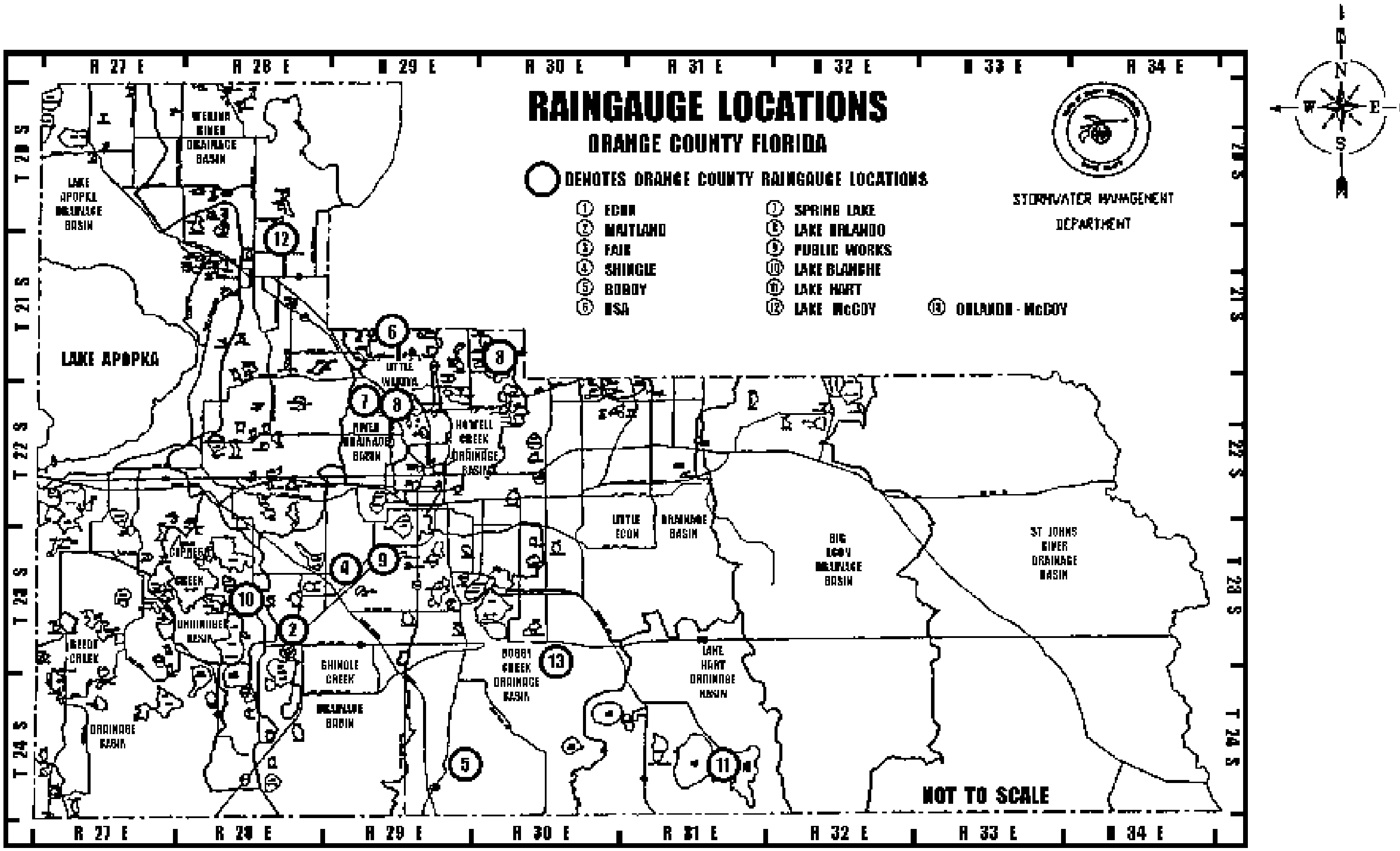


Figure A-4. Rain gauge locations. (Reprinted courtesy of the City of Orlando, FL)

For the 2-, 10-, and 25-year, 24-hour design storm events the Soil Conservation Service Type II Florida modified rainfall distribution (also called Type III) was selected based on the requirements of OUSWMM. The 100-year, 72-hour rainfall distribution was taken from the SFWMD permit manual. Rainfall intensities were then generated for each design storm.

Soil Types and Capabilities

Soils data are used to evaluate stormwater runoff, infiltration, and recharge potential for pervious areas. Information on soil types was obtained from the National Resources Conservation Service (NCRS), formerly the Soil Conservation Service (SCS). Each soil type has been assigned to a soil association, a soils series, and to one of the four Hydrologic Soil Groups (HSG) designated A, B, C, or D. HSG A is comprised of soils having very high infiltration potential and low runoff potential. HSG D is characterized by soils with a very low infiltration potential and a high runoff potential. The other two categories fall between the A and D soil groups.

For the Lake Hart study area, the majority of the soils types are within Smyrna-Bassinger-St. Johns soil association which are characterized by nearly level, poorly drained, and very poorly drained soils that are sandy throughout. The soils in the vicinity of Lake Nona, Red Lake and Buck Lake are classified as part of the Smyrna-Pomello-Immokalee association which are nearly level and have poorly drained soils to very well drained soils that are sandy throughout.

The predominant soils series within these subbasins include Sanibel Muck which has a depth to seasonal high groundwater table between zero and one foot and Smyrna Fine Sands which has a depth to seasonal high groundwater of one foot above the ground surface to one foot below the ground surface. The remainder of the soils are classified as part of the Pomello Fine sands which have a depth to seasonal high groundwater table between two and 3.5 feet or the St. Johns Fine Sands which have a depth to seasonal high groundwater table between zero and one foot.

Soil infiltration rates were taken from the NRCS Soil Survey for Orange County, FL based upon the soil hydrologic group. The RUNOFF Block of SWMM uses both soil storage and infiltration rates. Soil capacity (or soil storage) is a measure of the amount of storage (in inches) available in the soil type for a given antecedent moisture condition. The average antecedent moisture condition (AMC II) was used for all design storm analyses. Soil capacities were estimated based on available depth-to-water-table data and the use of equations as outlined in the SFWMD manual which uses equations developed by the NRCS. The high water table and low infiltration capacity conditions were considered in the best management practice (BMP) evaluations in subsequent sections to ensure that chosen alternative would function properly.

The Horton soil infiltration equation was used to simulate rain water percolation into the soil. The Horton equation uses an initial infiltration rate to account for moisture already in the soil, a maximum infiltration rate, and a decay infiltration rate. Additionally, a total maximum infiltration depth is computed based on the moisture capacity of the soil. In this study, the maximum depth was determined from the information provided in the Soil Survey of Orange County which documents seasonal high water tables or depths to the impervious layer (first impermeable boundary condition).

Once these infiltration parameters were computed and calibrated for each HSG, area-weighted parameter values were computed based on the percent of each HSG within a catchment. Detailed information on the use of the Horton infiltration equation is described in the SWMM 4.3 users manual.

Table A-2 lists the global infiltration parameters used to calculate the hydrologic input data used in this study. The global Horton infiltration equations presented in Table A-2 resulted in peak water surface elevations similar to those predicted by the Federal Emergency Management Agency (FEMA). This is based on CDM experience with over 30 stormwater management programs in Florida, including extensive calibration and verification to historic storms.

Table A-2. Global Horton Infiltration Parameters

Hydrologic Soil Group	Maximum Infiltration Rate (in/hr)	Minimum Decay Rate (in/hr)	Decay Rate (1/sec)	Maximum Soil Storage (in)
A	14.0	0.75	0.000556	5.4
B	10.0	0.50	0.000556	4.0
C	7.0	0.25	0.000556	3.0
D	5.0	0.10	0.000556	1.4

In order to manage the volume of data required to generate the SWMM RUNOFF data sets, spreadsheets were developed to semi-automate the process. Flow path data, land use data (including percent imperviousness), soil data, and tributary area measurements for each subbasin were input into a spreadsheet. The spreadsheet calculated area-weighted averages using the global Horton infiltration parameters and the hydrologic data to generate subbasin information that could be directly input to the SWMM RUNOFF data set.

Overland Flow Parameters

The RUNOFF module of SWMM uses overland flow data in the form of width, slope, and Manning's roughness coefficient to create a physically based overland flow runoff plane to route runoff to conduits and storage for further routing. The overland flow length (L) is the weighted-average travel length to the point of interest. The need for weighting becomes apparent when considering areas with odd geometry where a long, thin portion of the area may bias the hydraulic length. For ponded areas, the point of interest chosen was the centroid of ponding. For areas where ponding does not occur, the point of interest is the outflow from the area. Overland flow length is used to better estimate subbasin width for the RUNOFF overland flow routing by use of the equation:

$$A = LW$$

where:

A	=	subbasin area (sq. ft.)
L	=	overland flow length (ft.)
W	=	overland flow width (ft.)

Overland flow slope is the average slope over the hydraulic length and is calculated by dividing the difference in elevation by the hydraulic length. Length and slope information were obtained from 1985 aerial photogrammetry one-foot topographic data. These data were augmented by available subdivision plans and survey data.

Land Use and Impervious Areas

Land use data are used to estimate impervious areas for use in runoff calculations. Existing land use for the portion of the Lake Hart basin annexed by the City was obtained from 1985 aerial photography (1 in = 200 feet), 1995 aerial photography, and as-built information provided by the major property owners within the study area.

The majority of the study area consists of undeveloped lands (55%), wetlands (24%), and water bodies (15%). The remaining six percent of the total is a mixture of low density residential, golf course, commercial and major road land uses. Of the major property owners within the study area, only Lake Nona has constructed phases of their development plan.

The estimate of future land use was compiled from information provided by each of the major property owners within the basin and from information provided by the City of Orlando Planning Department. The developable land in the basin is projected to become low density residential (17% of study area), medium density residential (17% of study area), and supporting industrial/commercial land uses (12% of study area). The balance of the developable land (9%) is planned for schools, high density residential, golf courses and open space.

Using the existing and future land use data and the source maps, the percentage of each land use category within each subbasin was determined. Note that the future land use scenario represents a combination of City of Orlando information and the desires of the major property owners within the study area. The City has not adopted a future land use plan for this area.

The percent imperviousness of each subbasin is one of the parameters used by the SWMM RUNOFF model to determine the volume and rate of surface water runoff. For this study, a percent imperviousness value for each of the eleven land use categories was determined. A summary of the eleven land use categories is presented in Table A-

3. Additionally, the table lists the percent of Directly Connected Impervious Area (DCIA) and the percent of Non-DCIA (NDCIA) assigned to each land use category. The DCIA represents all the impervious surfaces which are directly connected to the stormwater system. The NDCIA represents the impervious surfaces that have a pervious buffer between them and the stormwater system.

Hydraulic Parameters

PSWMS (refer again to Figure A-2) for the Lake Hart basin consists of a series of interconnected lakes, streams, and wetlands that discharge to 10 different discharge points from the study area. There are 15 miles of open channels/interconnected wetlands (51 model segments), 33 structure crossings (e.g., culverts, bridges), and 35 existing storage areas representing lakes and depressional areas. Additional detention ponds were modeled for future land use. Characteristic data of this system were obtained from as-built drawings, field reconnaissance, one-foot contour topographic maps, and survey.

A necessary task of any stormwater master plan is the creation of a simplified representation of the actual system for input into the stormwater models. This task typically begins with the development of a model schematic which also aids in checking input data and interpreting output data. An overall RUNOFF/EXTRAN existing model schematic of the PSWMS for the entire Lake Hart study area is shown in Figure A-5. The schematic shows the hydrologic unit load points for inflow, conveyance channels, and structures, as well as the storage and linking junctions. It also illustrates how the RUNOFF and EXTRAN programs were set up to simulate each area's runoff hydrograph and the routing of the runoff through the stormwater management system. Identification numbers for various system elements are also shown on the schematic. The schematic provides a quick reference for correlations between the actual physical situation and the modeled system.

Table A-3. Imperviousness by Land Use Category

Land Use Category		Impervious[1] (%)	DCIA[2] (%)	NDCIA[3] (%)	Pervious (%)
1.	Forest, Open, & Park	1	1	0	99
2.	Agricultural & Golf Courses	1	1	0	99
3.	Low Density Residential	25	12.5	12.5	75
4.	Medium Density Residential	35	25	10	65
5.	High Density Residential	65	55	10	35
6.	Institutional	50	45	5	50
7.	Industrial	80	80	0	20
8.	Commercial	90	90	0	10
9.	Wetlands	100	100	0	0
10.	Water bodies	100	100	0	0
11.	Major Roads	98	98	0	2

Notes:
1) Total Impervious Area
2) Directly Connected Impervious Area (DCIA)
3) Non-Directly Connected Impervious Area (NDCIA)

Structures/Facilities

A major component of this study was the inventory of the stormwater management structures along the PSWMS. This information forms the foundation for the model representation of the hydraulic system. The hydraulic characteristics of the structures and facilities in the Lake Hart study area were collected from design drawings of improvements (e.g., culverts, bridges, detention ponds) that have occurred within the study area.

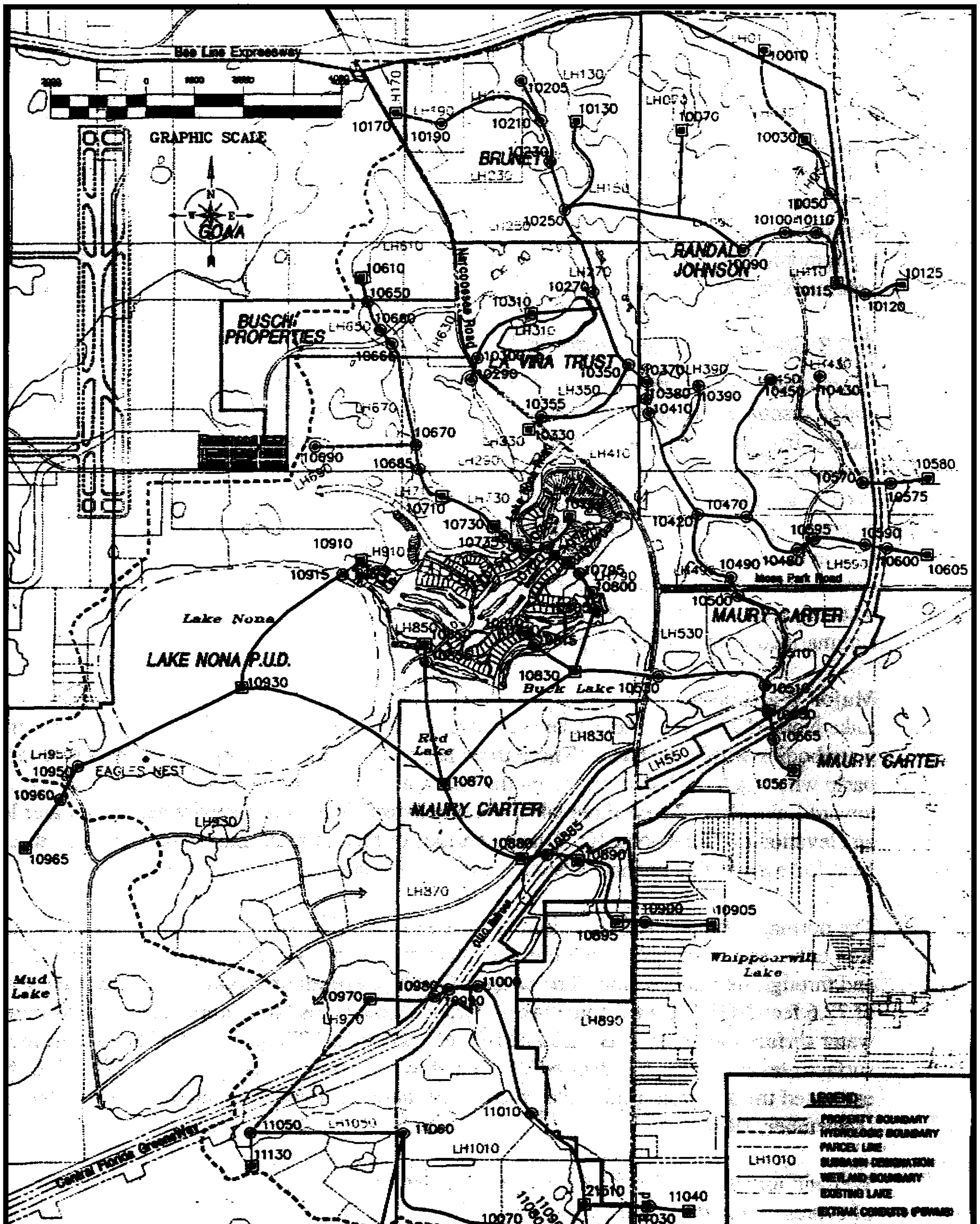


Figure A-5. Existing PSWMS nodal schematic map. (Reprinted courtesy of the City of Orlando, FL)

Stage-Area Relationships

Stage-area information was developed by planimetering topographic contours for major depressional areas which could not be uniformly incorporated into channel/wetland cross sections. This process was done to more accurately reflect floodplain storage. The same procedure was applied to the existing detention ponds. Stage-area relationships for existing facilities were obtained from topographic data shown on the as-built plans provided by the property owners within the basin. The volume of storage was internally calculated by stormwater models using the trapezoidal method.

Stage and Discharge Data

A desirable component of any water resources investigation is the availability of measured stages and/or discharges at selected points of interest, or the availability of calibrated hydrologic/hydraulic models from the area to serve as a "reality check" or verification. Stages and/or discharges are used in conjunction with known rainfall amounts/distributions and other hydrologic/hydraulic conditions to calibrate and verify models. These calibrated and verified models can then be used in evaluations of present problem area solutions or future conditions planning. Data in at least hourly intervals are often desired so that relatively short-term, yet potentially damaging, flood peaks can be predicted and planned for. For the Lake Hart basin, there are limited stage data and no discharge data available for use in the master planning process. The data that are available are summarized in the following paragraphs.

Lake Nona (575 acres), Red Lake (120 acres), and Buck Lake (115 acres) are the three major water bodies within the basin. These three lakes collect the majority of stormwater runoff from the basin which is then discharged from the lakes into a series of streams and wetlands that meander toward Lake Hart. These three lakes become hydraulically connected when their water level exceeds an elevation of 75.5 ft-National Geodetic Vertical Datum (NGVD). During periods of high rainfall, Lake Nona will also discharge into Mud Lake through a channel system located on the southwest side of the lake.

The normal water surface elevations and the seasonal high water surface elevations for Lake Nona, Red Lake, and Buck Lake were obtained from the Orange County Lake Index and through field inspection. The index reports a normal water elevation of 77.6 feet-NGVD for the three lakes. Orange County also took nine random measurements of the water surface elevation in Buck Lake between the years 1970 and 1975. The highest recorded water surface elevation was 77.8 feet-NGVD which was recorded on July 1, 1974. The FEMA also estimated the 100-year peak water surface elevation for these three lakes to be 79.6 feet-NGVD.

Wetland jurisdiction limits extend from the lake's open water body landward to where the dominance of cypress (*Taxodium distichum*), bay (*Gordonia lasianthus*), and tupelo trees (*Nyssa* sp.), ferns (*Osmunda* spp.) and shiny lyonia (*Lyonia lucida*) disappear. Upland areas include the canopy tree layer dominated by slash pine (*Pinus elliottii*), scrub live oak (*Quercus geminata*), and turkey oak (*Quercus laevis*), while saw palmetto

(*Serenoa repens*) dominate the understory. Extending the seasonal high water line and normal pool elevations landward would provide a reasonable wetland boundary around each lake. Hydric soils and hydrologic indicators would also need to be assessed to confirm the wetland jurisdiction line.

Biological indicators of wetland water levels were also used to approximate the normal pool and seasonal high water elevations at five sites within the Lake Hart basin. This was done using SWFWMD guidelines. The wetland jurisdictional determination methodologies implemented by Florida Department of Environmental Protection, and St. Johns River Water Management District (SJRWMD), and U.S. Army Corps of Engineers were also used to determine plant community zonation (i.e., obligate, facultative and facultative upland plant species) and to approximate temporal water inundations and conditions.

Using these guidelines, hydric soils characteristics, hydrophilic vegetation, and other biological information were compared with known topographic elevations to estimate normal pool and seasonal high water levels. No water level recorders or staff gages were present or were installed. The results of the field inspection for the five sites are summarized in Table A-4.

Table A-4. Field Estimated Normal Pool and Seasonal High Water Elevations

Site No. (invert)	Normal Pool (feet-NGVD)	Seasonal High (feet-NGVD)	Existing Water Level (feet-NGVD)	Indicators Used
1	78.1	78.6	77.3	Stain line Moss line
2	74	75.4	73.3	Stain line Moss line
3	76.9	77.7	76.4	Stain line Moss line
4	79	80	78.7	Stain line
5	73.1	75.1	72.4	Stain line Moss line

The results of the biological indicators at the five sites indicate that the maximum difference between the normal pool and seasonal high water elevations range from 0.5 feet to two feet. Various constrictions (e.g., inadequately sized culverts, culverts in poor condition, or inverts above than the 100-year flood event) may cause flow constrictions. The biological indicators provide fluctuation patterns, not duration.

The biological results provide a difference of water level fluctuation indicators for specific wetland species that adapt to prolonged inundation (i.e., adventitious roots and epiphytic algae) or are intolerant to sustained inundation (foliose lichens). Facultative and obligate plant indicators that occur along the landward extent of the wetlands can assist in the determination of the normal pool and seasonal high water levels. Many aquatic plants occur in specific horizontal zones along the slope and the changing water levels. Each species has adapted to a specific inundation period (duration). These hydrologic factors were used to differentiate the water distribution pattern and the extent of wetlands around each lake.

Floodplains and Floodways

A floodplain is the area inundated, or flooded, by a particular rain or tidal event. Floodplains are usually described by their frequency of occurrence (e.g., 25-year or 100-year). FEMA establishes nationwide flood levels and flood insurance standards. The FEMA flood insurance study (FIS) for Orange County, FL and associated Flood Insurance Rate Maps (FIRMs) identify portions of the Lake Hart basin annexed by the City as flood prone and provide estimates of the 100-year flood stages in order to provide guidance for home building and road elevations. For this study, available data were compiled in order to estimate stormwater flood boundary conditions for subsequent evaluations.

The City of Orlando requires that a Floodplain Development Permit be obtained for any development activities for any building or structure located in an area of special hazard. The general requirements for the permit application require that the applicant submit drawings to scale showing the nature, location, dimensions, and elevations of the area in question; existing and proposed structures; fill; storage or materials; and drainage facilities. Specifically, the following information is required:

- Base flood elevation (100-year flood)
- Habitable flood elevation
- Nonresidential floodproofing elevation
- Floodproofing certification
- Alteration of watercourse

Once this information is received, the City Engineer will review the application for compliance and issue a permit as appropriate. The City Engineer's review includes notification of other applicable regulatory agencies prior to any alteration or relocation of a watercourse, the verification of flood and structure elevations, determination of whether a building or development is within an Area of Special Hazard based on the applicable FEMA FIS and accompanying maps, and advise an applicant whether or not a Letter of Map Amendment or Revision from FEMA is required.

OUSWMM also has requirements for development in the floodplain. For example, encroachment will be allowed in the 100-year floodplain with compensating storage. All proposed developments within the 100-year floodplain as delineated on an official FIRM or as determined by the City Engineer need to comply with these requirements:

- City will establish the 100-year/24-hour base flood elevation

- If the area is not in a 100-year flood prone area, an analysis will be done to determine the 100-year elevation

- The design storm event to be used to establish the 100-year on-site elevation shall be a 100-year/72-hour event of 14.4 inches of rainfall

- The minimum finished floor elevation shall be one foot above the 100-year elevation

- Floodproofing may be substituted for elevating finished floor elevations for commercial and industrial developments

- Compensating flood storage must be provided for all floodwater displaced by development below the elevation of the 100-year/24-hour flood (generally, between the 100-year flood elevation and the wet season water table)

- Compensating storage may be claimed in retention/detention ponds when they are above maintained water elevations and they can be inundated during the 100-year flood.

- Off-site increases in flood stage will not be allowed by encroachment within a floodway.

Details on each of these summaries can be found in the appropriate chapters of the City Code and OUSWMM.

Water Quality Parameters

The following paragraphs discuss state surface water classifications, historical water quality data in the study area, trends exhibited by the data, and the methodology used to estimate nonpoint source pollutant loads. Data from the EPA's STOrage and RETrieval (STORET) database are included as appropriate.

Selection of Water Quality Loading Factors

In order to meet the objectives of the Lake Hart MSMP, pollutants that may affect water quality were identified and quantified. This section identifies stormwater related-pollutants in the study area and describes the methodology for determining appropriate event mean concentrations (EMCs) for use in the WMM.

Identification of Pollutants

The major sources of pollutants in a watershed are typically stormwater runoff from urban and agricultural areas, discharges from wastewater treatment plants (WWTPs) and industrial facilities, and contributions from improperly installed or maintained septic tanks. Stormwater runoff pollution and septic tank loadings have been historically referred to as nonpoint source pollution (NPS). A WWTP or industrial discharge is typically referred to as point source pollution because it releases pollution into streams at a discrete point. The Lake Hart MSMP targets the pollutants which are most frequently associated with stormwater including:

1. Sediment
   Total suspended solids (TSS)
   Total dissolved solids (TDS)

2. Oxygen demand
   Biochemical oxygen demand (BOD)
   Chemical oxygen demand (COD)

3. Nutrients
   Total phosphorus (TP)
   Dissolved phosphorus (DP)
   Total Kjeldahl nitrogen (TKN)
   Nitrate + nitrite nitrogen (NO_3+NO_2)

4. Heavy metals
   Lead (Pb)
   Copper (Cu)
   Zinc (Zn)
   Cadmium (Cd)

Estimates of the annual loads of these pollutants are required as part of the National Pollution Discharge Elimination System (NPDES) stormwater permitting analysis.

Selection of Stormwater Pollution Loading Factors

The nonpoint pollution loading module of WMM computes nonpoint pollution loads based on factors which relate local land use patterns and rainfall and percent imperviousness in a watershed to pollutant loadings. Nonpoint pollution loading factors (e.g., pounds/acre/year) for different land use categories are based upon annual runoff volumes and EMCs for different pollutants. The EMC is a flow-weighted average concentration and is defined as the sum of individual measurements of stormwater pollution loads divided by the storm runoff volume. Selection of EMCs factors depends upon the availability and accuracy of local monitoring data, as well as the effective transfer of literature values for nonpoint pollution loading factors to a particular study area. Reviewed here are monitoring data collected throughout Florida, as well as available literature values for estimating event mean concentrations for use in the Lake Hart MSMP.

Over the past 15 years, nonpoint pollution monitoring studies throughout the U.S. have shown that "per acre" discharges of urban stormwater pollution (e.g., nutrients, metals, BOD, fecal coliforms) are positively related to the amount of imperviousness in the land use (i.e., the more imperviousness the greater the nonpoint pollution load) and that the EMC is relatively consistent for a given land use. Soil types affect hydrology more than EMC, especially in areas dominated by impervious surfaces.

Land Use Load Factors

Recommended EMCs for the urban land use categories (residential, commercial, and industrial) in this plan are based upon a detailed analysis of available monitoring data recently collected under the EPA NPDES Part II Stormwater Permit application process. The process was conducted between November 1990 and May 1993 for over 34 NPDES municipal stormwater applications throughout the country including the states of Florida and Georgia. As part of the permit application process, representative stormwater outfalls were monitored in cities and counties with populations greater then 100,000. These "representative" outfalls typically discharged stormwater from areas with predominantly residential, commercial, or industrial land uses. Each outfall was monitored and sampled during a minimum of three separate storm events. The analysis included a total of 98 storm events that were monitored by selected cities and counties under the Florida Stormwater NPDES permitting process. Previously, the EPA sponsored Nationwide Urban Runoff Program (NURP) monitored stormwater pollution from urban areas in about 80 storm events in Tampa during 1978-1983.

Under the NPDES permitting process, flow-weighted composite samples were collected during storm events according to detailed sampling protocols prescribed by the EPA. Samples were analyzed for about 140 pollutants including those targeted for the Lake Hart MSMP. Statistical analyses of available NPDES data were used to determine appropriate EMCs for watershed management applications. Data from the City of Orlando NPDES monitoring sites were included in this analysis.

Some citrus and cattle growing/pasture land use exists or has existed in the study area. The pasture land use is in the northwest portion of the study area and the citrus is in the southeast. These two land uses are not well monitored nor documented for water quality in the literature. In particular, pasture EMCs can range dramatically if cattle are allowed to free range through streams and wetlands for water and forage. EMCs for total P can range from 0.3 mg/l to 1.0 mg/l or higher.

Total N can range from 1.45 mg/l to over 5 mg/l. Therefore, the most applicable central Florida values were used for these land uses to estimate existing land use pollutant loadings from these highly variable sources.

For central and south Florida, provides estimates of stormwater EMCs based on a literature review of monitoring studies performed at various sites in Florida. Dade County also prepared a literature review of selected EMC values to be used in the Dade County Stormwater Management Master Plan.

Open/Nonurban Land Use Load Factors

The only open/nonurban monitoring site included in the Florida NPDES sites analyzed was monitored by Sarasota County. This site did not include cattle pasture/growing or citrus.

Water Bodies

The primary sources of pollution to water bodies are runoff from upstream areas and pollutants associated with precipitation falling on the water surface. Since pollution discharged from upstream areas is already accounted for by the other land use category loading factors, loading factors for water bodies consider only the pollution derived from precipitation.

Urban atmospheric monitoring studies performed under NURP and other studies have documented that there is a pollution load associated with precipitation. Pollutant loading factors for water bodies were derived from the Tampa NURP atmospheric monitoring studies and a report containing a compilation of atmospheric deposit data. The loading factors used in this plan differ from those used in the Lake Hart MSMP based on an update of more recent and extensive data.

Major Roads

Highway runoff data reported by the Federal Highway Administration (FHWA) were considered for application to the major highway land uses in Florida watersheds. The FHWA study analyzed stormwater runoff monitoring data obtained at 31 highway sites covering a total of 993 separate storm events. Highway stormwater runoff data were collected under several previous studies during the past 10 to 15 years. Also, many of the previous FHWA monitoring studies were performed during periods when the use of leaded gasoline was more prevalent than today. These studies demonstrated that highway runoff may contain solids, metals, nutrients, oil and grease, bacteria, and other pollutants.

Recommendation of Stormwater Pollutant Loading Factors

From the databases described above, EMCs obtained from water quality monitoring studies completed in the state of Florida were used in this evaluation. These EMC values were compared with those obtained from studies throughout the eastern United States. Based on this comparison, the final EMC values were selected. These EMC values represent the best available information (most recent up-to-date database) and are applicable for pollutant load estimates in the City of Orlando. Table A-5 presents the recommended event mean concentrations and impervious percentages for the Lake Hart MSMP. Listed with each pollutant group is the reference source for these recommended EMCs.

Table A-5. Event Mean Concentrations and Impervious Percentages Recommended for the Watershed Management Model

Land Use Category	Avg. Percent Imp.	Oxygen Demand and Sediment (mg / L)					Nutrients (mg / L)					Heavy Medals (mg / L)				
		BOD	COD	TSS	TDS	SOURCE	TP	DP	TKN	NO23	SOURCE	Pb	Cu	Zn	Cd	Source
1. Forest, Open and Park	1.00%	1	51	11	100	A,B	0.05	0.004	0.94	0.31	A	0.000	0.000	0.000	0.000	B
2. Agriculture and Golf	1.00%	4	51	55	100	A,B	0.34	0.23	1.74	0.58	A	0.000	0.000	0.000	0.000	B
3. Low Density Residential	25.00%	15	71	27	286	C	0.44	0.33	1.34	0.63	C	0.002	0.009	0.051	0.002	C
4. Medium Density Residential	35.00%	9	65	59	59	C	0.45	0.27	1.77	0.27	C	0.013	0.007	0.057	0.001	C
5. High Density Residential	65.00%	8	53	42	141	C	0.20	0.09	1.03	0.67	C	0.011	0.022	0.065	0.001	C
6. Institutional	50.00%	7	50	41	114	C	0.15	0.08	1.24	1.05	C	0.012	0.018	0.079	0.001	C
7. Industrial	80.00%	14	83	77	130	C	0.28	0.20	1.47	0.40	C	0.023	0.024	0.132	0.001	C
8. Commercial	90.00%	8	53	42	141	C	0.20	0.09	1.03	0.67	C	0.011	0.022	0.065	0.001	C
9. Wetlands	100.00%	5	51	5	100	A,B,E	0.19	0.10	1.10	0.40	A,B,E	0.006	0.003	0.005	0.000	A,B,E
10. Waterbodies	100.00%	3	22	5	100	D,E	0.17	0.09	1.10	0.20	E	0.006	0.003	0.005	0.000	E
11. Major Roadways	98.00%	11	99	121	189	C	0.40	0.15	1.51	0.34	C	0.039	0.022	0.189	0.002	C

SOURCES:

A: "Estimation of Stormwater Loading Rate Parameters," Harvey H. Harper, 1992, Table 21.
B: Nationwide Urban Runoff Program (NURP), 1983.
C: NPDES Part II Stormwater Permit Applications for the Cities of Jacksonville, St. Petersburg and Orlando, and the Counties of Palm Beach and Sarasota, 1992-93.
D. "Washington Metropolitan Area Urban Runoff Demonstration Project," Northern Virginia Planning District Commission, January 1983, Table 24.
E: Mean concentrations reported for wetfall monitored as part of the Tampa NURP study and Mote Marine data compilation.

NOTES:

1. Dissolved - P concentrations for wetlands and Watercourses / Waterbodies are generally 55 percent of the recommended total - P concentration (Harper, 1992; Florida NPDES data, 1992 - 1993).
2. TKN and NO2 + NO3 concentrations for the non-urban land use categories were assumed to be 75 percent and 25 percent, respectively, of the recommended total - N concentration (Florida NPDES data, 1992 - 1993).
3. Averages reported are based on parametric statistics with a lognormal distribution.
4. Concentrations reported below the detection limits were assumed to 50% of the detection limits for the statistical analysis.
5. Golf courses were not explicitly included in the NPDES monitoring networks.

WMM converts the EMCs described above into nonpoint pollution loading factors (expressed as pounds/acre/year) based on the runoff volume for each land use within a watershed. Pollution loading factors vary by land use and the percent imperviousness associated with each land use. The pollution loading factor M_{LU} is computed for each land use (LU) based on the EMCs presented in Table A-5 using the following equation:

$$M_L = EMC_L * R_L * K$$

Where:

M_{LU}	=	loading factor for land use LU (lb/ac/year)
EMC_{LU}	=	event mean concentration in runoff from land use LU (mg/l). EMC_L varies by land use and by pollutant
R_{LU}	=	total average annual surface runoff from land use LU (in/year)
K	=	0.2266, a unit conversion constant ((lb-l)/(mg-ac-in))

The total annual pollution load from a watershed is computed by multiplying the pollutant loading factor by the acreage in each land use and summing for all land uses.

Delivery Ratio/Travel Time

Wet-weather travel times on the order of 24 hours or more are typically required to achieve significant decay of pollutants during instream transport. While in-stream settling occurs on an annual basis, the resuspension of sediments in streams is likely to carry pollutants downstream. Therefore, in order to provide more conservative estimates of the nonpoint source loads, a delivery ratio of 100 percent was assigned to all areas within the City of Orlando for pollutants suspended in the water column.

Point Source Discharge

Pollutant loadings from point source dischargers, such as regional WWTPs, are usually estimated to determine the relative contributions of point versus nonpoint pollution loadings. The Lake Nona wastewater treatment facility is within the study area. However, it is not considered to be a point source discharge because effluent from the WWTP is discharged into a holding pond that is used for slow-rate spray irrigation at the golf course so that it does not directly discharge into the PSWMS.

BMP Pollutant Removal Efficiencies

WMM applies a constant removal efficiency for each pollutant to all land use types to simulate treatment BMPs. Recommended pollutant removal efficiencies for retention basin, detention basin, and swale BMPs are discussed below.

The design of retention systems is generally based on a specified diversion volume. Relying on extensive field investigations and simulations using 20 years of rainfall data, average yearly pollutant removal efficiencies were estimated for fixed diversion volumes for onsite (small) watersheds, as presented in Table A-6. The diversion depth is the depth of runoff water which must be stored and percolated from the total upstream drainage area that discharges to the retention pond.

The EPA NURP study monitored several wet detention ponds serving small urban watersheds in different locations throughout the U.S. For wet detention ponds with significant average hydraulic residence times (e.g., two weeks or greater), average pollutant removal rates were on the order of 40 to 50% for total-P and 20 to 40% for total-N. For other pollutants which are removed primarily by sedimentation processes, the average removal rates were as follows: 80 to 90% for TSS; 70 to 80% for lead; 40 to 50% for zinc; and 20 to 40% for BOD or COD.

Pollutant removal efficiencies for dry extended detention ponds are based on settling behavior of the particulate pollutants. Table A-6 summarizes average pollutant removal efficiencies for dry extended detention ponds based on settling column data and field monitoring data. Settling column data from NURP studies and from the FHWA study were evaluated to establish the removal efficiencies for TSS and metals.

Removal efficiencies for the nutrients were determined by evaluating the results of two field monitoring studies of dry extended detention ponds in the metropolitan Washington, D.C. region. These efficiencies are applied to the percentage of total annual pollutant washoff captured for treatment in the extended dry detention pond.

The removal efficiencies summarized in Table A-6 for swales represent swales designed for infiltration and capture of 80 percent of the annual runoff volume. These efficiencies are based upon NURP findings and CDM experience. Finally, the pollutant removal rates for retention swale pre-treated upstream of a wet detention pond are based on retaining the first 0.25 inches over the tributary area coupled with full wet detention treatment.

Surface Water Quality Classifications

Section 403.021 of Florida Statutes declares that the public policy of the state is to conserve the waters of the state to protect, maintain, and improve the quality thereof for public water supplies, for the propagation of wildlife, fish, and other aquatic life, and for domestic, agricultural, industrial, recreational, and other beneficial uses. It also prohibits the discharge of wastes into Florida waters without treatment necessary to protect those beneficial uses of the waters. Furthermore, Congress, in Section 101(a)(2) of the Federal Water Pollution Control Act, as amended, declared that achievement by July 1, 1983 of water quality sufficient for the protection and

Table A-6. Average Annual Pollutant Removal Rates for Retention Basin, Detention Basin and Swale BMPs (Note: All values are percent.)

	Extended Dry Detention [1]	Wet Detention [2]	Retention[3]	Swales [4]	Retention Swales With Wet Detention[5]
BOD5	30	40	90	30	76
COD	30	40	90	30	76
TSS	90	90	90	80	96
TDS	0	40	90	10	76
Total-P	30	50	90	40	80
Dissolved-P	0	70	90	10	88
NO2+NO3	0	30	90	40	76
TKN	20	30	90	40	72
Cadmium	80	80	90	65	92
Copper	60	70	90	50	88
Lead	80	80	90	75	92
Zinc	50	50	90	50	80

NOTES:

1. Extended dry detention basin efficiencies assume that the storage capacity of the extended detention pool is adequately sized to achieve the design detention time for at least 80 percent of the annual runoff volume. For most areas of the United States, extended dry detention basin efficiencies assume a storage volume of at least 0.5 inches per impervious acre.

2. Wet detention basin efficiencies assume a permanent pool storage volume which achieves average hydraulic residence time of at least two weeks.

3. Retention removal rates assume that the retention BMP is adequately sized to capture at least 80 percent of the annual runoff volume from the BMP drainage area. For most areas of the United States, the required minimum storage capacity of the retention BMP will be in the range of 0.50 to 1.0 inch of runoff from the BMP drainage area, but the required minimum storage capacity should be determined for each location.

4. Source: California Stormwater Best Management Practice Handbooks, (CDM, et. al., 1993). These efficiencies are applied to the percentage of total annual pollutant washoff captured for treatment in the extended dry detention pond BMP.

5. This efficiency reflects removal efficiencies for series BMPs with 0.25 inches of retention swale pre-treated upstream of a wet detention pond.

propagation of fish, shellfish, and wildlife, as well as for recreation in and on the water, is an interim goal to be sought wherever attainable. Congress further states, in Section 101(a)(3), that it is the national policy that the discharge of toxic pollutants in toxic amounts be prohibited.

Therefore, the present and future most beneficial uses of all waters of the state have been designated by the FDEP using the classification system set forth in Chapter 62-302, of the Florida Administrative Code. These water quality standards and associated criteria have been established to protect designated uses which are:

1. OFW Outstanding Florida Waters, which include waters in state and federal parks, wildlife refuges, and other environmentally sensitive areas.

2. Class I: Potable Water Supplies.

3. Class II: Shellfish Propagation or Harvesting.

4. Class III: Recreation, Propagation and Maintenance of a Healthy, Well-Balanced Population of Fish and Wildlife.

5. Class IV: Agricultural Use.

6. Class V: Navigation, Utility, and Industrial Uses.

Accordingly, the FDEP has established minimum, general, and specific criteria for surface waters in the state. These criteria provide limits for various detectable sources of pollution (e.g., nutrients, metals, organics). Water quality data are needed to document adverse impacts to Water bodies/watercourses and flora/fauna. Stormwater generates nonpoint source pollutant loads which can degrade water quality. Traditionally, water quality data are collected in regular intervals (e.g., quarterly) to record ambient conditions in a given location. However, stormwater sampling is needed during specific storm events to properly monitor for the "flush" of pollutants in rivers and streams.

By using these water quality data, water classifications, and criteria, recommendations can be made regarding the BMPs to use to achieve the standards established for, or mitigate the adverse impacts to, the receiving body of water. The following sections discuss available water quality data and potential water quality trends in the study area. The receiving waters in this study area are Lake Hart, Red Lake, Buck Lake and Lake Nona which are designated as Class III waters.

Historical Water Quality Monitoring Data

Historical water quality data are available for Lake Nona, Red Lake, and Buck Lake. The following paragraphs present a brief summary of current water quality.

To measure water quality of Florida lakes, an index of bio-physical and chemical parameters (trophic classification system) has been developed. Lakes containing similar (cluster) analysis results of seven indicators (primary production (pp), chlorophyll a (CHA), total organic nitrogen (TON), total phosphorus (TP), Secchi disc transparency

(SD), conductivity (COND), and a cation ratio (CR) due to Pearsall (1922)) were classified into four trophic levels and ranked (Brezonik and Shannon, 1971). The trophic state index is delineated by numerical values into four classes: oligotrophic (0-49), mesotrophic (50-60), eutrophic (61-69), and hypereutrophic (70-).

The Orange County Environmental Protection Department conducted annual water quality studies for all the county lakes beginning in 1990 to the present. The department measures four of the original seven parameters: chlorophyll a (a component of algae), Secchi depth (water clarity or transparency), total phosphorus, and total nitrogen (nutrient indicators). As a natural lake ages (eutrophication), a shift from oligtrophic (few nutrients) to eutrophic (well nourished) conditions occurs. Industrial, agricultural, and urbanization activities around a lake accelerate this process. Table A-7 provides the annual trophic state index (TSI) results of the calculations which rank the Lake Hart basin.

The TSI results show that natural eutrophication has occurred basin wide. Each lake shows a slight increase in value during the five year study. Red Lake and Lake Nona have retained their oligotrophic status. Buck Lake and Lake Whipporwill have recently changed from oligotrophic to mesotropic conditions. Lake Hart has maintained a mesotrophic level being within five increments of the range. In contrast, the two oligotrophic lakes have no or minimum urbanization activities. Overall the water quality in Lake Nona, Red Lake and Buck Lake is good. The Orange County TSI survey showed that Lake Nona was ranked second out of 136 lakes, with Buck Lake 68, Lake Whipporwill 76, and Lake Hart 109. The results are summarized in Table A-8.

Biological quality of selected lakes in Orange County were measured in 1994. Table A-9 provides the Diversity Index (a measurement of the variety of biological organisms which exists within a community), Equitability (a measurement of the distribution of the various types of biological organisms within a community and Taxa Richness (an average number of the species present at the site sampled.

Table A-7. The Annual Trophic State Index Results for the Lake Hart Basin

Lake Name	1990	1991	1992	1993	1994
Buck	45	--	54	50	50
Hart	53	50	56	57	58
Nona	30	20	15	28	22
Red	39	44	44	49	40
Whipporwill	34	38	52	46	51

Table A-8. 1994 Summary of Lake Secchi Disk Measurements, Chlorophyll-a Concentrations and Nitrogen and Phosphorus Concentrations in the Lake Hart Basin

Lake Name	Secchi Disk m	Chlor-a ug/l	NO_2-NO_3 mg/l	TKN mg/l	TN mg/l	TPO_4 mg/l	TSI Index
Buck	1.8	7.5	0.02	0.95	0.97	0.03	50
Hart	0.5	2.9	0.1	1.06	1.16	0.03	58
Nona	3.8	1.6	0.01	0.27	0.28	0	22
Red	2.3	3.5	0.02	0.64	0.66	0.02	40
Whipporwill	1.3	9.5	0.02	0.55	0.57	0.02	51

Source: Orange County Environmental Protection.

The results of the lakes in Table A-9 reflect a moderate pollution condition (eutrophic) in comparison to other lakes in central Florida. The results of the next two lakes are outside the Lake Hart basin that show one lake with eutrophic conditions and one lake with oligotrophic conditions, respectively. Lake Rowena was sampled on January 13, 1993, had a TSI of 57, a Diversity Index of 1.38, an Equitability of 0.3, and a Taxa Richness of 12. Lake Wauseon was sampled on December 29, 1993 had a TSI of 30, a Diversity Index of 3.2, an Equitability of 0.52, and a Taxa Richness of 30.5.

Table A-9. Biological Quality of Selected Lakes in Orange County

Lake	Date	Diversity Index	Equitability	Taxa Richness
Hart	2/8/93	2.45	0.64	11
Whipporwill	2/8/93	2.52	0.67	12

Source: Orange County Environmental Protection.

Evaluation of Best Management Practices

Best Management Practices Considerations

Best Management Practices (BMPs) are techniques, approaches, or designs that promote sound use and protection of natural resources. Various types of BMPs are discussed extensively in Chapter 6 of the FDER Land Development Manual, 1989. This section summarizes alternatives which can be used to control flooding and avoid water quality problems.

Alternative Best Management Practices

BMPs that were considered for use in the Lake Hart basin MSMP are listed below where they are grouped as structural (constructed facilities) and non-structural (regulations or ordinances):

Structural Stormwater Controls

1. Extended dry detention ponds
2. Wet detention ponds (with and without retention swales)
3. Exfiltration trenches
4. Shallow grassed swales
5. Retention basins
6. Porous pavement
7. Water quality inlets
8. Underdrains and stormwater filter systems
9. Alum injection
10. Aeration
11. Skimmers

Non-Structural Source Controls

1. Land use planning
2. Public information programs
3. Stormwater management ordinance requirements
4. Fertilizer application controls
5. Pesticide use controls
6. Solid waste management
7. Street sweeping
8. Aquifer recharge and minimization of directly connected impervious area
9. Illicit connections (non-stormwater discharges) identification and removal
10. Control of illegal dumping
11. Erosion and sediment
12. Source control on construction sites
13. Operation and maintenance

The use of a specific BMP depends on the site conditions and objectives such as water quality protection, flood control, aquifer recharge, or volume control. In many cases, there are multiple goals or needs for a given project. Therefore, BMPs can be "mixed and matched" to develop a "treatment train." The treatment train concept maximizes the use of available site conditions from the point of runoff generation to the receiving water discharge in order to maximize water quantity (flood control), water quality (pollutant load reduction), aquifer recharge, and wetlands benefits.

The City currently applies the treatment train concept for wet detention facilities as described in OUSWMM. The runoff generated by the first inch of rainfall is stored in an off-line retention facility that is separate from the detention facility. Once the retention volume is exceeded, stormwater runoff flows into a separate detention facility for flood control where it is gradually discharged to receiving water as necessary. For the South East Annexation Area (SEAA), the City will consider alternative innovative options to meet the goals of OUSWMM. This is discussed in further detail in this “Evaluation of Best Management Practices.”

Figure A-6 and Figure A-7 show, respectively, a schematic flowchart of the treatment train concept and the City's "two pond" wet detention system.

Operation and Maintenance (O & M)

A recent survey by FDEP reported that nearly 70% of existing treatment facilities in Florida are not properly maintained and, therefore, do not provide the intended pollutant removal effectiveness. Because of this, one of the most effective non-structural BMPs is routine maintenance of existing treatment facilities. For publicly owned treatment facilities, routine maintenance and inspection should be considered for facilities that are within water quality sensitive basins. For the other "non-critical" areas, maintenance of treatment facilities may be considered on an as needed basis based on periodic inspection reports.

For privately owned facilities, maintenance is not typically performed by a municipality. There are several options that can be pursued by a municipality to help insure that proper maintenance is being conducted. These options may include a certification program initiated by a municipality that requires all approved private subdivision ponds to be recertified by the owner on a predetermined time interval. The re-certification may be done by a state certified/trained inspector or engineer. Enforcement of maintenance of privately owned facilities is one of the most difficult problems for a municipality. A potential enforcement measure is City intervention, after sufficient notification, where critical maintenance is done by the City and the cost of the maintenance is billed to the owner. Another option would be to consider stormwater utility credits for certified maintenance and rehabilitation.

Regional Versus Onsite Structural Best Management Practices

In much of the undeveloped portions of the City of Orlando, regional detention of flood control and water quality protection for relatively flat areas with high water tables appear to be the solution of choice because they provide the needed multiple benefits. The following discussion is provided for detention pond applications, which tend to be cost-effective where sited regionally.

Onsite Approach

In the case of future urban development, the onsite (also known as piecemeal approach to stormwater control) involves the delegation of responsibilities for BMP deployment to local land developers. Each developer is responsible for constructing a structural BMP at the development site to control nonpoint pollution loadings from the site. Detention pond BMPs provided onsite typically have contributing areas of 20 to 50 acres. The local government is responsible for reviewing each structural BMP design to ensure conformance with specified design criteria, for inspecting the constructed facility to ensure conformance with the design, and for ensuring that a maintenance plan is implemented for the facility. The onsite approach is illustrated in Figure A-8.

Regional Approach

The regional approach to stormwater control involves strategically siting regional structural BMPs to control nonpoint pollution loadings from multiple development projects. The front-end costs for constructing the structural BMP are assumed by the developer and/or the local government entity that administers the regional BMP plan. BMP capital costs can then recovered from upstream developers on a pro-rata basis as development occurs. Individual regional BMPs are phased in as development occurs rather than constructing all regional facilities at one time. Maintenance responsibility for regional structural BMPs can be assumed by the developer (or designee with certified maintenance bonds) or by the local government. The regional approach addresses concurrence for the entire watershed while the onsite approach does not address this issue. The regional approach is also shown in Figure A-8.

In developing stormwater and watershed management programs during the 1970s, local governments often elected to use the piecemeal approach because it required no advanced planning and, therefore, appeared relatively easy to administer. While the lack of planning requirements does give the piecemeal approach an up-front advantage, in comparison with the regional approach, the long term disadvantages outweigh this benefit.

A regional BMP system offers benefits that are equal to or greater than onsite BMP benefits at a lower cost. Most of the advantages of the regional approach over the onsite approach can be attributed to the need for fewer structural facilities that are strategically located within the watershed. The specific advantages of the regional approach are summarized below

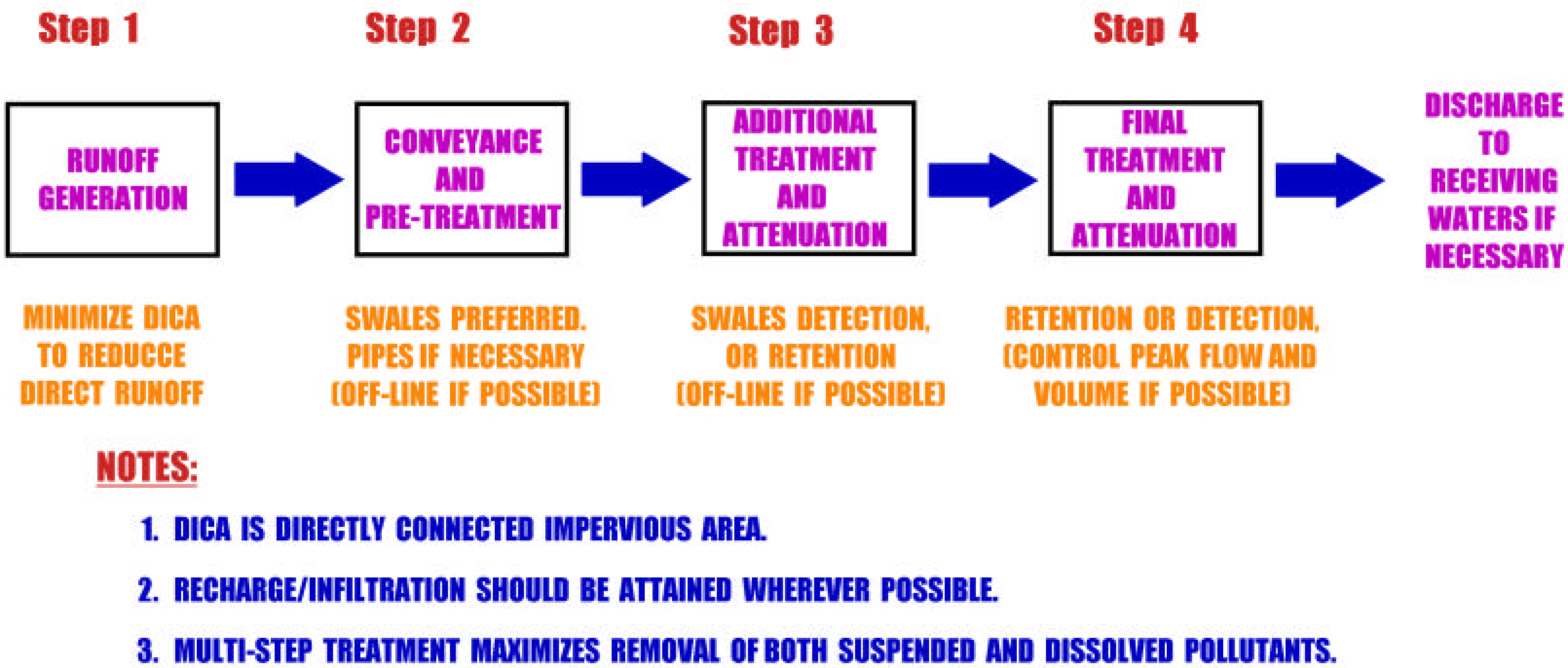


Figure A-6. Best management practice "treatment train" concept (Reprinted Courtesy of the City of Orlando, FL).

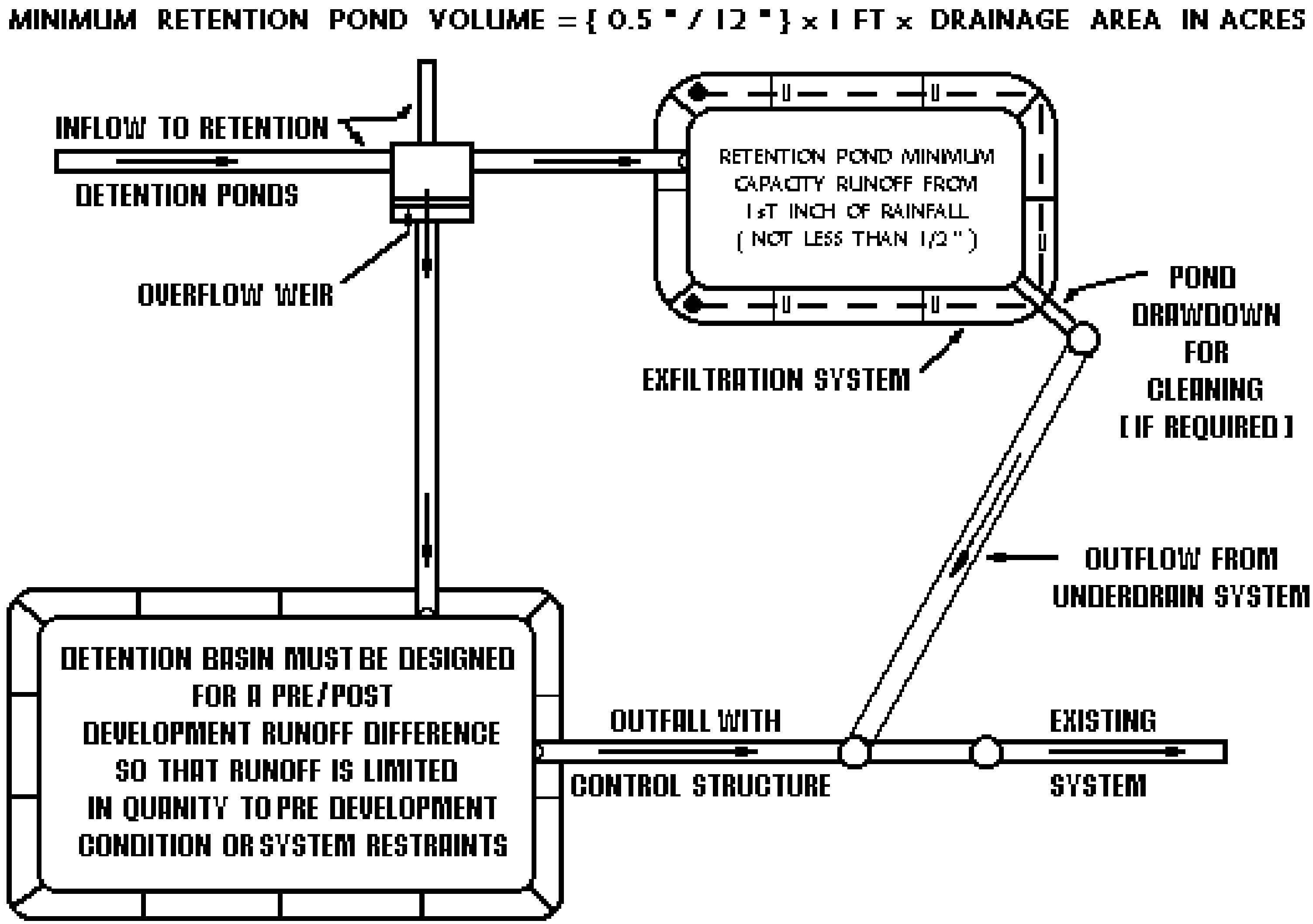


Figure A-7. Design for retention/detention facilities (Reprinted Courtesy of the City of Orlando, FL).

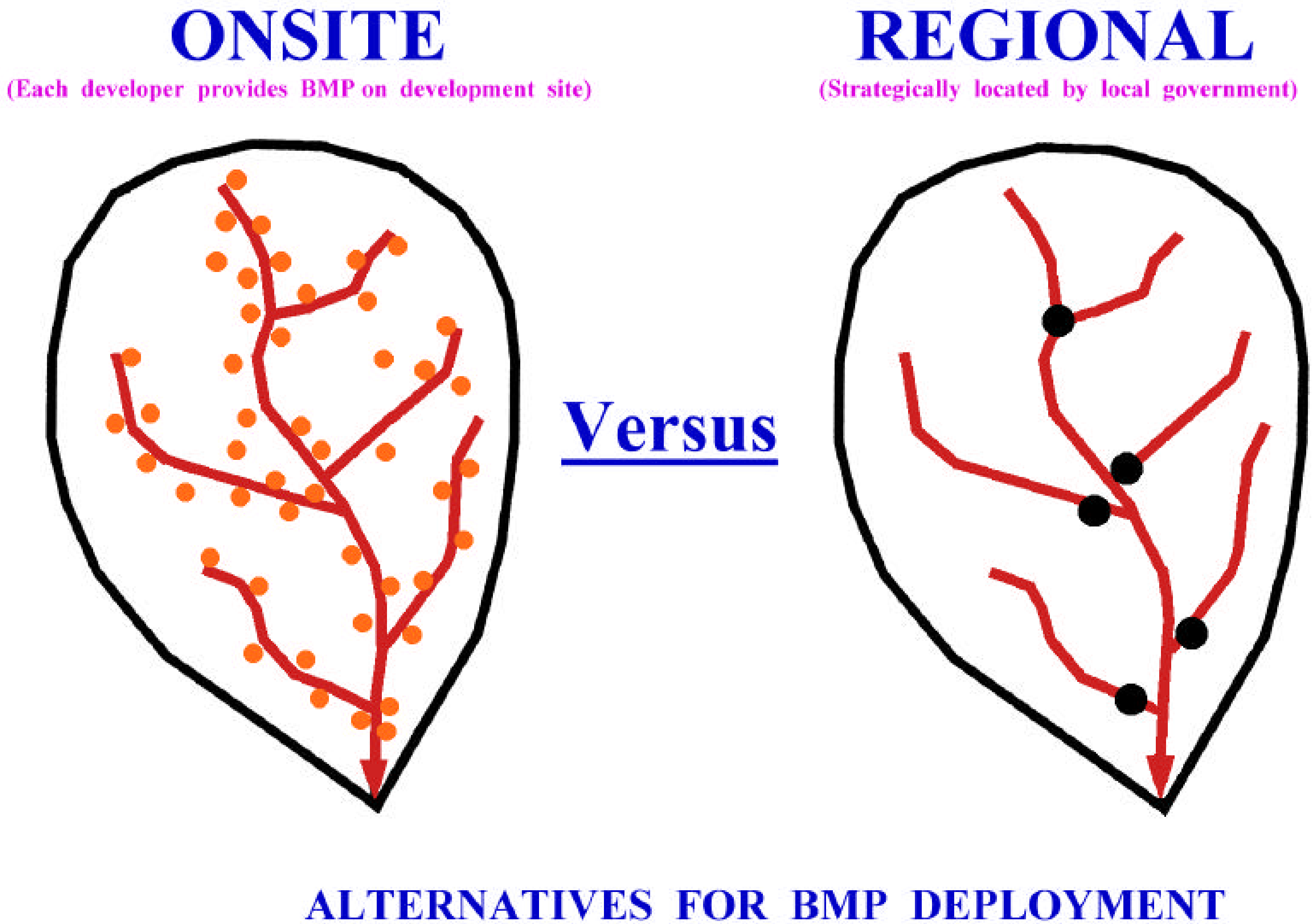


Figure A-8. Onsite versus regional best management practices (Reprinted Courtesy of the City of Orlando, FL).

- Reduction in maintenance costs: Since there are fewer stormwater detention facilities to maintain, the annual cost of maintenance programs are significantly lower. Moreover, because the regional detention facility recommended in the master plan can be designed to facilitate maintenance activities, annual maintenance costs are further reduced in comparison with onsite facilities. Examples of cost saving design features that are typically only feasible at regional BMP facilities include: access roads that facilitate the movement of equipment and work crews onto the site (by comparison, detention facilities implemented under the onsite approach are often located in residential backyards), additional sediment storage capacity (e.g., sediment forebay) to permit an increase in the time interval between facility clean-out operations, and onsite disposal areas for sediment and debris removed during clean-out.

- Greater reliability: A regional BMP system will be more reliable than an onsite BMP system because it is more likely to be maintained. With fewer facilities to maintain and design features that reduce maintenance costs, the regional BMP approach is much more likely to result in an effective long-term maintenance program. Due to the greater number of facilities, the onsite BMP approach tends to result in a large number of facilities that do not get adequate maintenance and, therefore, soon cease to function as designed. Many municipalities start off with the onsite approach but eventually switch to the regional approach to address the lack of maintenance of the onsite systems and to increase the overall effectiveness of the stormwater management program. Regional facilities, however, cannot be so large that incremental water quality protection is lost. For instance, if a regional detention facility is at the bottom of a 10 square mile basin, no water quality protection would be provided to the upstream rivers and streams as urbanization occurs. This could be detrimental to the existing plants and wildlife species. Another problem with an excessively large regional facility is the impact of the facility on existing wetlands. In rural areas, an excessively large pond would inundate large wetland areas which would make permitting of the structures extremely difficult. Experience shows that a regional pond should be limited to a 100 to 600 acre tributary area.

- Opportunities to manage existing non-point pollution loadings: Nonpoint pollution loadings from existing developed areas can be affordably controlled at the same regional facilities that are sited to control future urban development. This is because the provision of additional storage capacity to control runoff from existing development in the facility's contributing area is reasonable in cost as a result of economies-of-scale. By comparison, the costs of retrofitting existing development sites with onsite detention BMPs to control existing nonpoint pollution loadings may be prohibitively expensive.

- Fairness to land developers: Land developers recognize that economies-of-scale available at a single regional BMP facility should produce lower capital costs in comparison with several onsite detention facilities. They also tend to prefer the regional BMP approach because it eliminates the need to set aside acreage for an onsite facility other than pretreatment and conveyance to the regional pond. This could permit an increase in the number of dwelling units within the development site while still providing sufficient stormwater management. The additional cost of a pond sized for future development can be passed on to the developer. Developers can "buy" into the regional system and eliminate on-site BMP requirements, thus minimizing cost to the public. Regional facilities also offer the ability to maximize mining of fill material which will be necessary in the Lake Hart basin.

- Multi-purpose uses: Regional facilities can often be landscaped to offer recreational and aesthetic benefits. Jogging and walking trails, picnic areas, ball fields, and canoeing or boating are some of the typical uses. For example, portions of the facility used for flood control can be kept dry, except during floods, and used for exercise areas, football or soccer fields and softball or baseball diamonds. Wildlife benefits can be provided in the form of islands or preservation zones which allow observation of nature within the park schemes. Gradual swales can also be worked into the park concept to provide pretreatment around paved areas, such as parking lots or access roads. Figure A-9 illustrates a typical multi-purpose stormwater facility.

Best Management Practices Implementation Considerations

In determining the best stormwater management facility or combination of facilities (treatment train), various factors need to be considered. Examples are:

- Physical constraints or requirements of the site such as permeability of the soil, the location of the wet season high water table, and the amount of land available on the site to construct the facility.

- Permitability of the facility or facilities.

- Needed benefits to solve problems and guide future development in a given area.

- Benefits provided by the facility such as control of peak discharge for flood control, reduction in the total volume of discharge, groundwater recharge, erosion control, wetlands management, reduction of pollutant loads to receiving waters, and/or optimized maintenance. Table A-10 lists requirements and benefits that can be used as a guide in the selection of a stormwater BMP type.

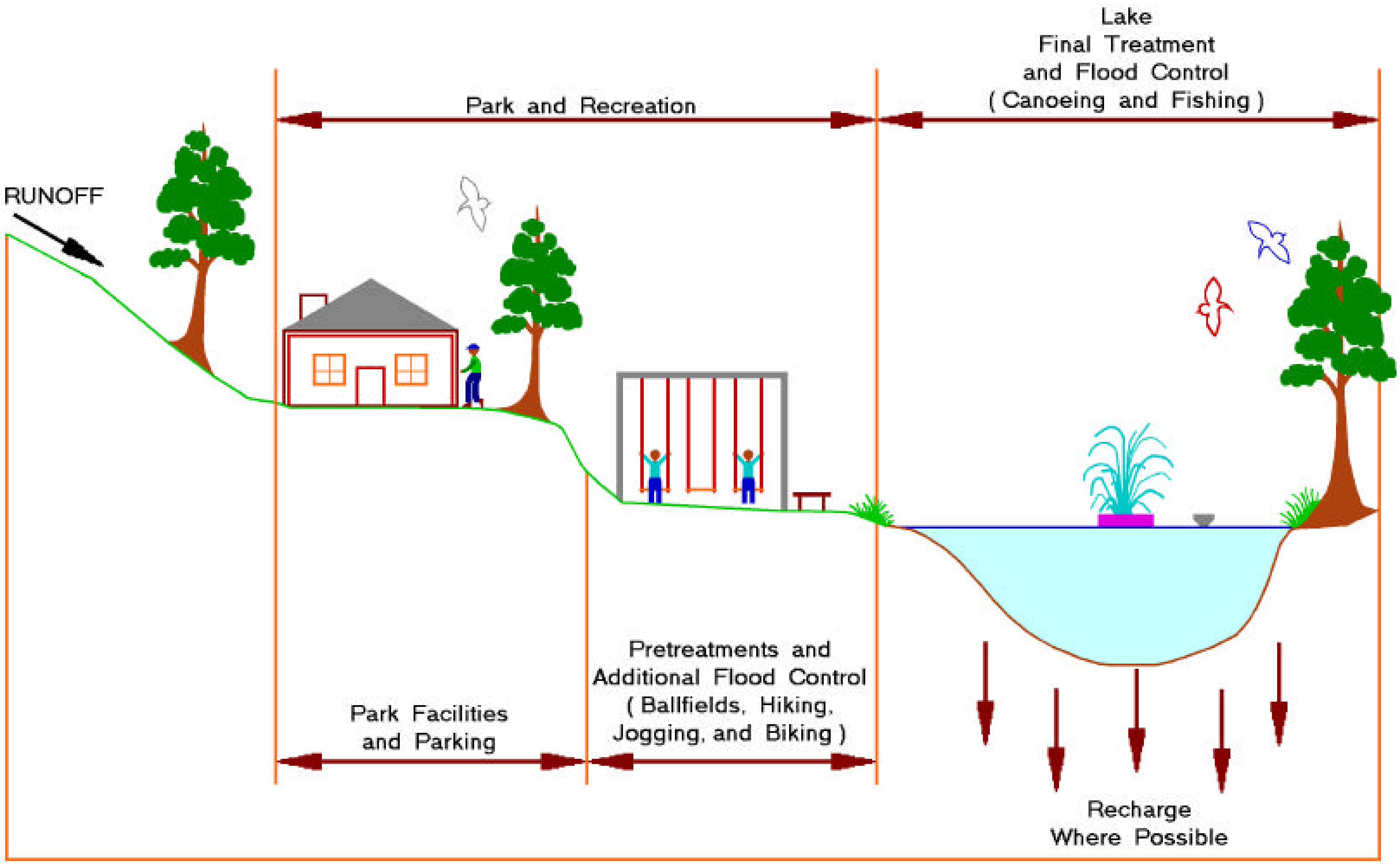


Figure A-9. Typical multi-use stormwater facility (Reprinted Courtesy of the City of Orlando, FL).

Table A-10. BMP Selection Features:: Requirements Versus Benefits

BEST MANAGEMENT PRACTICE					
Extended Dry Detention Ponds	Wet Detention	Exfiltration Trenches	Shallow Grassed Swales	Retention Basins	Filtration
Requirements:					
1. Available Space	1. Available Space	1. Limited Space Available	1. Moderate to Limited Space Available	1. Available Space	1. Available Space
	2. Water Table at or Near Pond Normal Pool Level	2. Water Table > 2 Ft Below Trench Bottom	2. Water Table > 1-2 Ft Below Swale Bottom	2. Water Table > 2-3 Ft Below Basin Bottom	2. Minimal Base Flow
	3. Relatively Impermeable Soils	3. Highly Permeable Soils	3. Permeable Soils		
Benefits:					
1. Peak Discharge Control 2. Load Reduction for Suspended Pollutants 3. Multiple-Use Park Areas	1. Peak Discharge Control 2. Load Reduction for Dissolved and Suspended Pollutants 3. Aesthetic Permanent Pool and Fountain	1. Aquifer Recharge 2. Pollutant Load Reduction On-Line	1. Peak Discharge Control 2. Volume Discharge Control 3. Aquifer Recharge	1. Peak Discharge Control 2. Volume Discharge Control 3. Aquifer Recharge	2. Aquifer Recharge
	4. Wildlife Habitat 5. Multi-Use Park Areas		4. Pollutant Load Reduction Off-Line or On-Line 5. Pre-Treatment	4. Pollutant Load Reduction Off-Line or On-Line 5. Multiple-Use Park Areas	

Recommended Best Management Practices

Introduction

The previous section titled “Evaluation of Best Management Practices” presented a discussion of various BMP types, and their benefits and limitations. The recommended BMPs, as discussed in the section, are proposed to become the foundation for a South East Annexation Area (SEAA) Stormwater Management Manual (SWMM). As already noted, two general categories of controls can be implemented to improve or enhance stormwater runoff with respect to water quality and water quantity (flooding). Structural controls are constructed facilities that treat, store, or convey stormwater runoff. Non-structural controls, on the other hand, focus on the prevention of pollution and the reduction of runoff. This section presents the recommended BMP treatment train.

The BMPs discussed in the previous section were screened for applicability to the Lake Hart basin study area based on site constraints, cost-effectiveness, efficiency, maintenance requirements, and current OUSWMM guidelines. Since the basin is largely undeveloped with few existing problems, the focus of the alternative analysis was planning regional facilities for the control of runoff from future development (quality and quantity control). The Lake Hart basin has the following physical characteristics:

1. Relatively flat terrain.
2. High groundwater table.
3. Need for flood storage.
4. Need for treatment of solids and soluble pollutants.
5. Need for fill for development and improvement projects.

Because of these physical characteristics, wet detention BMPs were considered to be the most appropriate control measures to meet the program goals.

Based on the LOS goals of the program, system constraints, SFWMMD permitting requirements, the Narcoossee Road improvements, and developer needs, a BMP Treatment Train has been formulated with three major components: DCIA minimization, pretreatment (0.25 inches) and regional wet detention ponds.

OUSWMM requires that wet detention facilities use a two pond system. The first pond uses retention to provide water quality treatment and the second separate pond uses detention for flood control. Because of the high groundwater table in the Lake Hart basin developable areas (typically one to two feet below the ground surface), deeper retention pond systems (two to four feet) may not function as desired. Therefore, shallow pretreatment practices may be incorporated into landscaping swales and lot grading plans as an alternate. The BMP treatment train would build upon the foundations of OUSWMM by providing nearly equivalent innovative technology considerations for areas with these site constraints:

- Lakes as receiving waters.
- Karst topography.

- Twenty-four percent of the basin is comprised of wetlands.

The BMP treatment train for the Lake Hart basin would consist of several pretreatment practices primarily within the secondary stormwater management system in series with regional wet detention ponds protecting the PSWMS. This innovative approach will achieve both the water quantity and water quality goals of OUSWMM while allowing for a cost-effective regional facility concept for future development. In addition, this concept is consistent with annexation agreements between the City, County, and local land owners. The recommended BMPs (pretreatment and wet detention) for the Lake Hart basin are discussed below.

Pretreatment Best Management Practices

The pretreatment BMPs are a series of structural and non-structural controls that will provide a reduction in runoff volumes and/or pollutant loads from urbanized areas prior to their discharge into the regional wet detention ponds and the downstream wetlands. The structural pretreatment BMPs will provide treatment for approximately 0.25 inch of runoff over the tributary area. Structural controls include retention swales with raised inlets to allow overflows, wet detention ponds, and oil-water separators for individual areas. Non-structural BMPs include reducing DCIA by diverting rooftops and portions of driveways and parking lots to shallow, grassed, or landscaped swale areas, and runoff pollutant source reduction methods -- many of which are voluntary but would help to achieve benefits. The recommended pretreatment BMPs are discussed below.

Minimization of Directly Connected Impervious Area

Minimizing DCIA involves ensuring that as much runoff as possible from impervious areas is routed over relatively large pervious areas and, in some cases, choosing an alternative surface to pavement or concrete that allows for some degree of infiltration. Figure A-10 is an illustration of a parcel that has been modified to convert a portion of the DCIA into non-directly connected impervious area by rerouting the roof gutters over the lawn (properly graded between houses). A portion of the DCIA could be converted to pervious area by using a porous surface.

Landscaped Swales and Grass-Lined Swales

Landscaped swales should be used around parking lots, houses, and other structures. The swales will provide pretreatment and also provide conveyance to larger secondary or primary stormwater management systems. Properly designed swales are useful for proper grading around houses as well as detention/retention prior to discharge into a

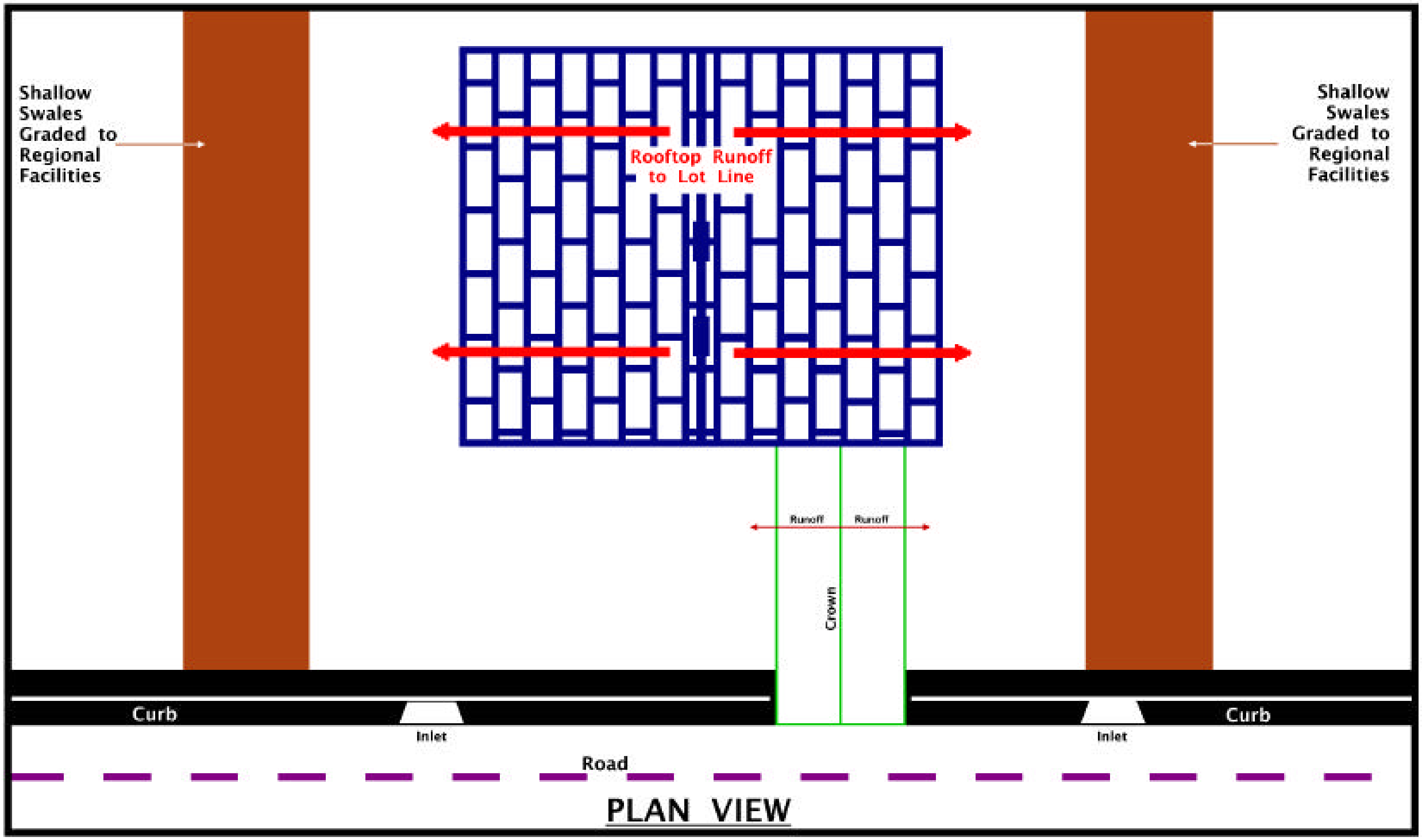


Figure A-10. Minimization of directly connected impervious area and use of grass lined swales. (Reprinted Courtesy of the City of Orlando, FL).

secondary or primary system. Fill from the shallow swale area may be used elsewhere on the property to improve the grading plan. Landscaped swales would typically be 0.5 to 1.0 foot deep and should have side slopes no steeper than 4:1 (H:V), with side slopes of 6:1 or greater being less noticeable and more attractive.

Grass-lined swales should be constructed around parking lots and commercial centers as recessed planters for landscaping. The swales could be part of the landscaping and incorporate raised inlets into the design, which will allow for the initial 0.25 inch retention volume for pretreatment. Although groundwater tables in the developable area are generally within one to two feet of the surface, recovery times for retention volumes of approximately 0.25 inch should be sufficiently small to allow the use of limited retention. Minimum infiltration rates of 0.1 inch/hour are expected to be advisable, allowing a relatively quick drawdown. Swales incorporated within commercial areas can enhance aesthetics and be used as credit towards green space and landscaping requirements. Figure A-11 shows an example of a landscaped swale with a raised inlet. Runoff will serve to reduce irrigation needs.

Curb Connections to Swales

Connections from the curbs to roadside swales should be provided to route street flow to grass-lined swales before discharge to the secondary or primary stormwater management system. Because roadway runoff may contain a greater pollutant load than runoff most other surfaces, providing swale pretreatment of roadway runoff will reduce pollutant loads to the regional ponds and improve the overall efficiency of the BMP treatment train. The swale space required for pretreatment of roadway runoff in roadside swales can be incorporated into OUSWMM green space requirements and be used to enhance the aesthetics of the roadways.

The connections between the curb and the swale can be implemented in two ways. The first method is to provide regularly spaced flumes in the curb as the connection to the swale. This method would be less expensive and will be aesthetically appealing. Another way, as illustrated in Figure A-12, is to provide a four to six inch diameter pipe approximately every 200 feet between the curb and the swale. This method may provide better erosion control at the edge of the curb by preventing water from flowing over the turf between the curb and the swale. The disadvantage to this method is the potential for clogging of the small pipes and thus the requirement for increased maintenance.

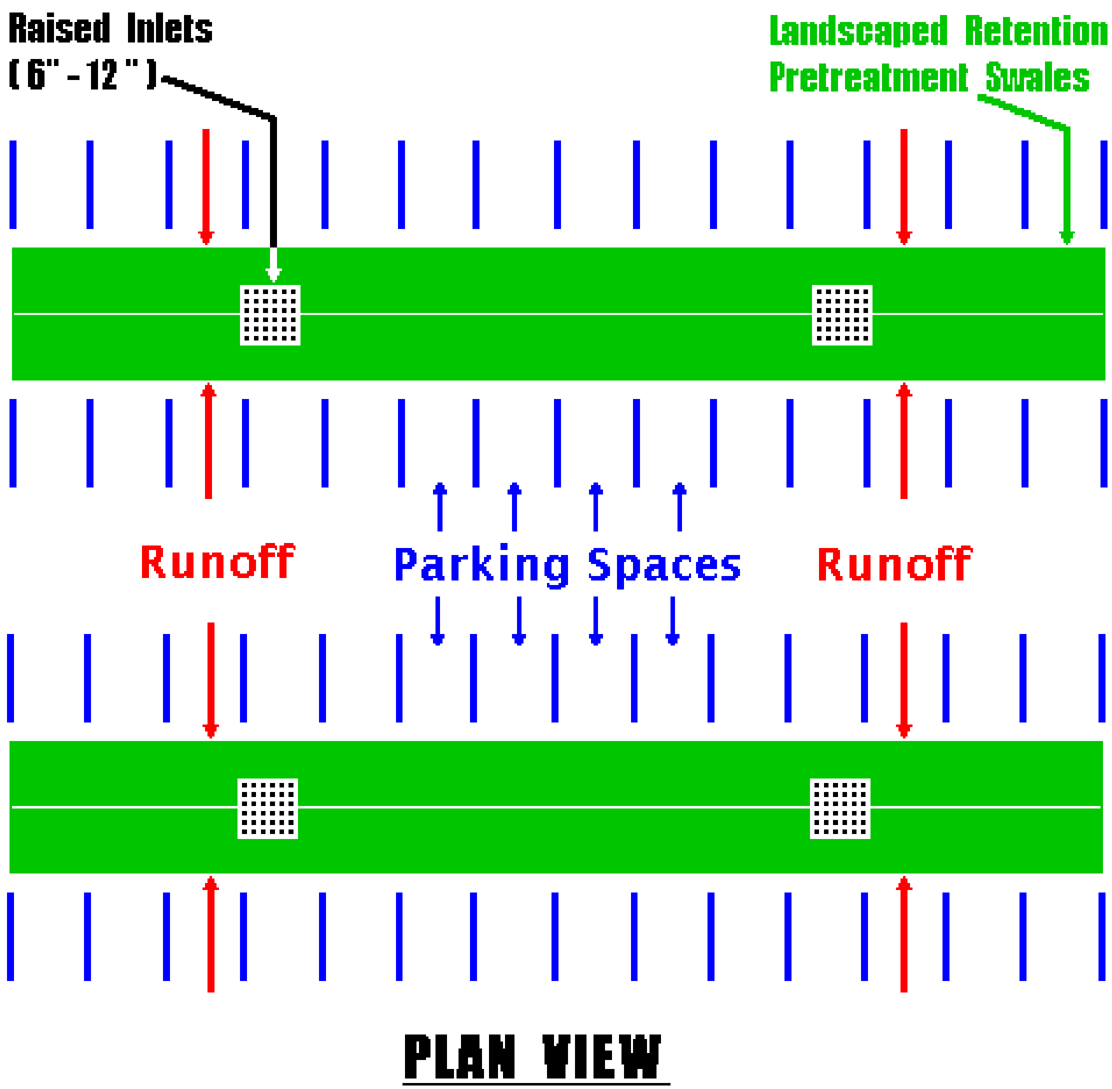


Figure A-11. Landscaped retention pretreatment swales with raised inlets (Reprinted Courtesy of the City of Orlando, FL).

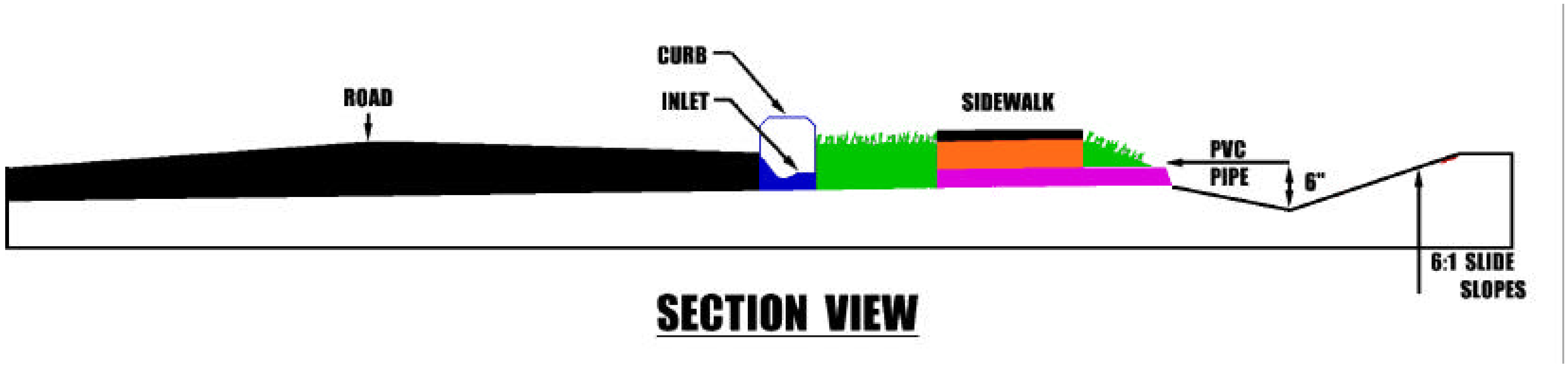


Figure A-12. Use of pipe to convey roadway runoff to roadside swale (Reprinted Courtesy of the City of Orlando, FL).

Capture Ratios of Swales

The Storage, Treatment, Overflow, and Runoff Model (STORM) was used to evaluate the effectiveness of the pretreatment swales at capturing a percentage of the annual runoff and, therefore, the annual pollutant volume. STORM is a continuous simulation model developed by CDM for the United States Army Corps of Engineers (USACOE) Hydrologic Engineering Center (HEC) that translates a continuous, long-term rainfall record (1942 through 1993 was used for this study) into a series of runoff events based on hydrologic conditions, routes the runoff through a "treatment facility," and calculates statistics on outputs such as runoff volumes and pollutant loads.

In the mode used for this analysis, the characteristics of the treatment facility were described by a storage volume(e.g., 0.25 inches) and a treatment rate. The treatment rate in this case is equal to the infiltration rate in the swale normalized to the total contributing area. Characteristic swales were established for both residential and commercial areas using the swale configuration previously discussed. Because there will be variability based on site conditions and application, a range of treatment rates and storage volumes around the expected values were used to establish the sensitivity to the results. Results from these simulations are shown in Figure A-13 for medium density residential areas. The average annual runoff volume capture ratio is approximately 60% for a 0.25 inch retention volume and typical soils in the area. Treatment efficiencies for the BMP treatment train were adjusted accordingly since the wet detention ponds would treat and attenuate about 40% of the average annual runoff volume.

Oil-Water Separators

Potential sources of high oil and grease, such as gas stations and light industrial land uses, should be required to provide either oil-water separation devices or off-line retention. Off-line retention offers additional pollutant removal benefits beyond oil and grease removal, provides additional volume control, and requires typical maintenance. However, off-line retention is also more space intensive and may result in groundwater contamination if sufficient quantities of pollutants are released into the retention basin. Oil-water separators require less space and initial capital expense. They need to be maintained at least monthly and offer some control of floating and settleable solids.

Sediment Forebays

Sediment forebays should be designed into the regional wet detention ponds. Forebays are designed to be easier to maintain than the rest of pond. The use of forebays will lower maintenance costs and extend the time between maintenance dredging of the remainder of the pond. Figure A-14 shows a typical forebay.

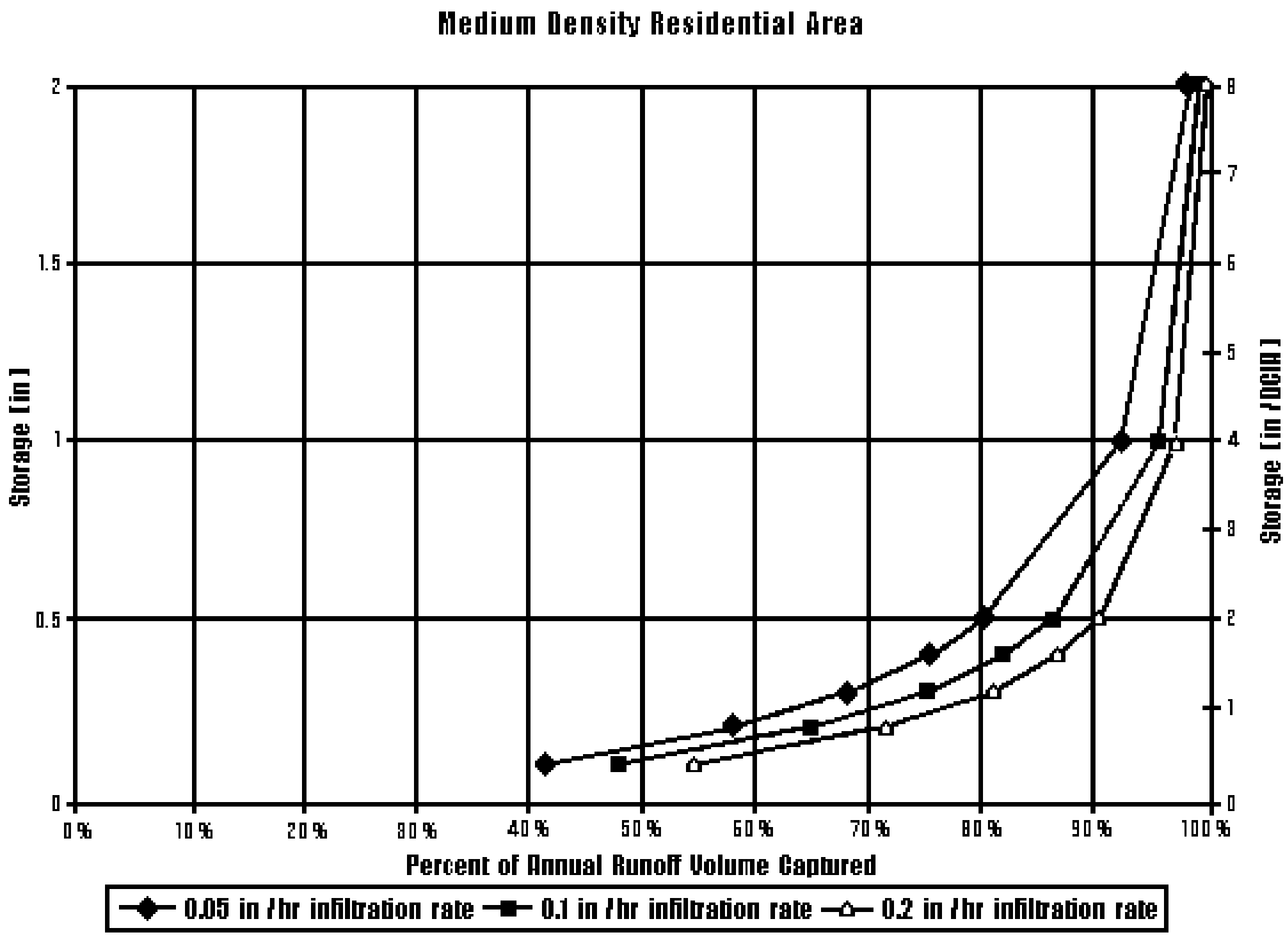


Figure A-13. Percent of annual runoff volume captured for medium density residential (Reprinted Courtesy of the City of Orlando, FL).

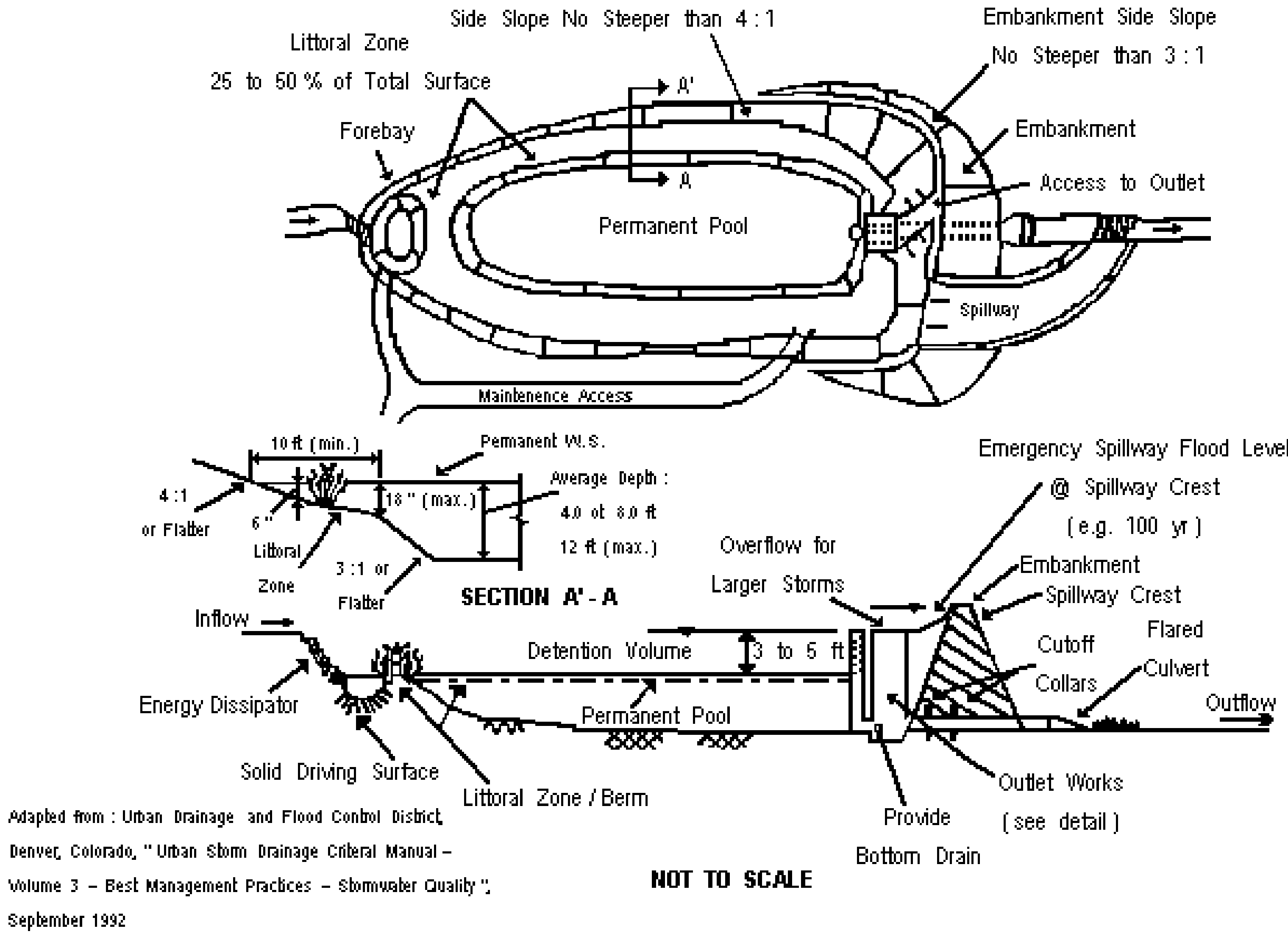


Figure A-14. Typical wet pond with forebay (Reprinted Courtesy of the City of Orlando, FL).

Source Reduction

Control of pollutants at the source of generation is a very effective and economical pretreatment BMP. Source reduction requests for illicit corrections and illegal dumping are needed for the EPA NPDES permit order. Source reduction relies almost entirely on the education of citizens living and working in the area. Examples of education programs for source reduction of pollutants are fliers instructing how to use the minimal amount of lawn fertilizer and pesticide and stenciled messages on storm drains.

Wet Detention Location and Sizing Criteria

The following paragraphs discuss the general criteria used to site the proposed regional facilities as well as the methodologies used to size them.

Regional Facility Location Criteria

A major component of this MSMP was the cooperative effort between the City of Orlando and private property owners during the siting of the proposed regional facilities. This was accomplished through a series of group and individual meetings with the major property owners and their engineers to discuss the advantages and disadvantages of each proposed regional facility location. Criteria discussed during these meetings included siting the regional facilities such that program goals of flood control, water quality protection, aquifer recharge and wetland protection could be achieved. In addition, other implementation considerations were incorporated, such as maximizing road frontage, developable property, waterfront property, and tributary area served. Additionally, accessibility of the regional facilities by maintenance crews was considered during the siting process. From an environmental perspective, the regional facilities were sited adjacent to wetlands (wherever possible) and conceptually designed with V-notched weirs that would discharge into the wetlands in such a manner that the existing wetlands would be preserved.

Coordination of the Narcoossee Road widening project with proposed development in the study area was also a key factor in siting the proposed regional facilities. There are potentially seven regional ponds that would provide stormwater management for both Narcoossee Road and surrounding proposed developments. By serving a dual purpose, fewer ponds would be required which represents capital operation and maintenance cost savings to both the City and private property owners.

Regional Facility Sizing Methodology

The proposed regional facilities were sized using the guidelines documented in the City of Orlando Urban Stormwater Management Manual (OUSWMM) and the SFWMD Management and Storage of Surface Waters (MSSW) Permit Information Manual Volume IV. A discussion of these guidelines and their application to wet detention is present below. Two volumes are used in sizing a wet detention system. They are the live pool (sometimes called treatment pool volume) and the permanent pool. Combined, these two components have a regulated discharge to detain water and settle pollutants to achieve the desired water quality goals.

Live Pool Volume

Chapter 5.2.1 of the SFWMD MSSW Permit Information Manual provides guidelines on determining the required treatment pool volume for a wet detention system. The requirements state that "wet detention volume shall be provided for the first inch of runoff from the developed project, or the total runoff of 2.5 inches times the percentage of imperviousness, whichever is greater". The same criterion is used in Chapter 2.8.4 of the OUSWMM. Therefore the live pool volume computed for each of the proposed facilities was determined using the following equations:

Maximum of

```
V SUB L ~ = ~ { R1*A*Ia } OVER { 12 ~ inch )foot }
```

```
V SUB L ~ = ~ { R2*A } OVER { 12 ~ inch )foot }
```

or

where:

V_L	=	Live pool volume (acre-feet)
R1	=	2.5 inches of rainfall
R2	=	1.0 inches of runoff
A	=	Tributary area (acres)
Ia	=	Average impervious area (percent)
	=	(NDCIA + DCIA)/100
NDCIA	=	Non directly connected impervious area (percent)
DCIA	=	Directly connected impervious area (percent)

Because of the high seasonal groundwater tables identified for the study area, the maximum treatment pool depth was assumed to be one foot above the permanent pool to ensure proper flood protection. This criterion became one of the key elements in determining the pond surface area requirements.

Live Pool Volume Bleed-Down Requirements

The criteria in the OUSWMM manual also requires that 50% of the live pool volume can be discharged in the first 60 hours following a storm event with total volume recovery occurring in 14 days. The bleed-down requirements presented in the SFWMD MSSW Permit Information Manual Volume IV (Chapter 7.2) are for a release of no more than 0.5 inches per 24 hours.

The SFWMD basis of review requires that bleed-down mechanisms be V-notches for wet detention systems. The discharge through a V-notch opening is a weir can be estimated by:

Q ~ = ~ 2.5*tan (2) 2) *H SUP { 2.5 }

where:

Q	=	Discharge (cfs)
2	=	Angle of V-notch (degrees)
H	=	Head on vertex of notch (feet)

Since SFWMD criteria specified that this bleed-down mechanism be sized to discharge one-half inch of detention volume in 24-hours, the following formula provides the required size:

2 ~ = ~ 2*tan SUP {-1} ~ {(0.492*Vdet)} OVER H SUP {2.5}

where:

2	=	V-notch angle (degrees)
Vdet	=	One-half inch of detention volume (acre-feet)
H	=	Vertical distance from weir crest to vertex angle (feet)

For the Lake Hart MSMP, the SFWMD criteria were used for sizing the V-notch control weirs.

Permanent Pool Volume

Chapter 2.8.4 of the OUSWMM manual lists the following requirements for the permanent pool volume:

- "The volume in the permanent pool (below the maintained water level) must be sufficient to provide a residence time of at least 14 days. This volume may be determined as 2-inches over the impervious portion of the drainage basin, plus ½-inch over the pervious portion of the drainage basin"

- "A littoral shelf shall be incorporated into the facility from maintained water level or a depth of 2.5 feet at a slope no steeper than 6:1"

- "The facility shall be configured such that the mean depth is 3 to 10 feet. Recommended depth ratios are:"

Percent Area	Depth, feet
< 10	> 8
50-70	4-8
25-50	0-4

Using these requirements, the permanent pool volume was calculated as follows:

```
Vp~ = ~ {[A*Ia*R3+A*(1-Ia)*R4]} OVER {12 ~ inch)foot}
```

where:

Vp	=	Required permanent pool volume (acre-feet)
A	=	Tributary area (acres)
Ia	=	Average impervious area (percent)
	=	(NDCIA + DCIA)/100
R3	=	2.0 inches of rainfall over the impervious area
R4	=	0.5 inches of rainfall over the pervious area

There are no specified permanent pool volume requirements identified in the SFWMD MSSW Permit Information Manual. However, the SFWMD has identified similar criteria to that in the OUSWMM for geometric considerations of a wet detention system (Chapter 7.4). A summary of these criteria are as follows:

- The facility must have a minimum wet detention surface area of 0.5 acres.

- The wet detention facility should have a 2:1 length to width ratio (applicant can request a waiver of this criteria if there is a single owner, or the entities involves have a full time maintenance staff with an interest in maintaining the areas for water quality purposes).

- The littoral area should be shallower than six feet as measured below the control structure elevation. The littoral area shall be 20% of the wet detention area or 2.5% if the total wet detention area (including side slopes) plus the contributing area. The SFWMD also recommends that 25 to 50% of the wet detention area be deeper than 12 feet.

- Side slopes shall not be steeper than 4:1.

- Bulkheads shall be allowed for no more than 40% of the shoreline length, plus compensating littoral zone must be provided.

For planning purposes, the required depth of the permanent pool for each facility was estimated for the OUSWMM criteria or as 70% of the area would have a depth of six feet and 30% of the area would have a depth of one foot which results in an average depth of 4.5 feet. Individual ponds could be constructed deeper to the SFWMD maximum values if additional fill is needed. This would provide a longer residence time. Aerating fountains are also recommended to control water quality (higher dissolved oxygen).

Flood Control Requirements

Chapter 2.9 of the OUSWMM lists the flood control requirements of the City. These requirements are summarized as follows:

- The additional volume of runoff generated by development shall be controlled and released at a rate not to exceed the peak rate for the site in the undeveloped condition. The design criterion shall be the 25-year/24-hour storm event.

- For landlocked primary basins, volumetric controls apply. The excess runoff from development for the 100-year/24-hour storm event shall be held on-site.

- Normally, the detention for flood control must be accomplished in an area separate from that used to provide pollution abatement. For the Lake Hart MSMP, this criterion was modified to include a second alternative by the City to allow single ponds with the pretreatment of 0.25 inches runoff onsite.

Chapter 2.10 of the OUSWMM addresses flood prone areas. Definitions included in this section include:

- The floodplain is the area inundated during the 100-year/24-hour storm event.

- The floodway is that portion of the floodplain which must be clear of encroachment in order to limit the increase in flood stage to one foot.

The requirements for flood prone areas as presented in this section are summarized as follows:

- Encroachment will be allowed within the 100-year floodplain, with compensating storage.

- All development within the 100-year floodplain established by FEMA or the City shall comply with the following:

- If the project is not within a 100-year flood prone area, an analysis shall be performed to establish the site's 100-year elevation.

- The design storm event to establish the 100-year onsite elevation shall be the 100-year/72-hour storm event.

- The minimum finished floor elevation shall be at least one foot above the elevation from the 100-year/24-hour storm, or at the maximum stage for the 100-year/72-hour storm.

- For commercial or industrial developments, flood proofing may be substituted for elevating the finished floor (careful consideration should be given prior to implementing this alternative).

- Compensating storage must be provided for all floodwater displaced by development below the 100-year/24-hour storm event. Compensating storage may be claimed in the retention/detention ponds provided it is above the maintained water elevations and berm elevations are such that the pond can be inundated during the 100-year storm and still provide 25-year flood protection.

- Off-site increases in flood stage and/or velocity will not be allowed by encroachment within a floodway. (The 100-year/72-hour design storm top width in flow should be considered as the floodway along the wetland tributaries.)

- A letter of map revision will be required for development within the defined FEMA floodplain.

Chapter 6 of the SFWMD MSSW Permit Information manual lists water quantity criteria. A summary of these criteria area is as follows:

- Offsite discharge rate is limited to rates not causing adverse impacts to existing offsite properties and historic discharge rates, rates determined in previous SFWMD permit actions, or rates specified in SFWMD criteria.

- Unless otherwise specified by SFWMD permits or criteria, a 25-year/72-hour storm event shall be used in computing offsite discharge rates. Alternate discharge rates can be requested from the SFWMD if adequate justification can be provided.

- Building floors shall be above the 100-year flood elevation as determined from the FEMA FIRM or from the 100-year/72-hour storm event. Lower elevations will be considered by the SFWMD for non-residential uses.

- In cases where flood protection of roads is not specified by local government, the 5-year/24-hour storm event shall be used for flood protection. The minimum roadway crown elevation shall be at least two-feet higher than the control elevation.

- No net encroachment into the floodplain, between the average wet season water table and that encompassed by the 100-year event, which will adversely affect the existing rights of others, will be allowed.

Based on these criteria, the regional facilities were sized so that peak flows and elevations from the 25-year/24-hour and 100-year/72-hour design storm events were not increased at any of the ten discharge points. This was accomplished using the stormwater model developed for this study.

Regional Stormwater System Review Considerations

A critical element in the implementation of the Lake Hart basin MSMP will be the review by the City of the stormwater facility design plans from developers to ensure that recommendations for the Lake Hart basin are being satisfied. Ultimately a detailed checklist should be prepared that will assist reviewers in determining if the recommendations are being met. The items listed below are an outline for a preliminary checklist to be filled in by the designer and used by the reviewers:

1. Basin number.
2. Tributary area (ac).
3. Land use and soil parameter consistency.
4. Pretreatment volume (ac-ft).
5. Pond treatment volume (live and permanent pools, ac-ft).
6. Forebay.
7. Pond flood volume (ac-ft, this can include the live treatment volume).
8. Connection to PSWMS (method).
9. Control structure (details).
10. Flow, stage, and velocity (summaries).

After the completion of this study, the checklist and more detailed statistics could be produced to provide the step-by-step outline needed for implementation.

Water Quality Results

Introduction

The Lake Hart basin MSMP included an evaluation of nonpoint source pollutant loads caused by land use changes and their associated BMPs. The nonpoint source pollution assessment was performed to estimate the annual average and seasonal stormwater pollutant loads for the twelve EPA NPDES indication parameters, including biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), total dissolved solids (TDS), total nitrogen (TN), total Kjeldahl nitrogen (TKN), total phosphorus (TP), dissolved phosphorus (DP), cadmium (Cd), copper (Cu), lead (Pb), and zinc (Zn). From this analysis, a base set of pollutant loads was established under existing land use conditions with the existing BMPs. Under future land use conditions, pollutant load projections are made with both the existing and proposed BMPs and compared to the existing loads. The relative changes in present and future pollutant load projections are used as an indicator of the potential for water quality impacts. This comparison then helps to identify the effectiveness of SFWMD and City criteria for controlling pollutant load increases as well as assisting in determining the level of control that will be required in the future.

Scenarios

Average annual nonpoint pollutant source loads from the study area were projected using the Watershed Management Model (WMM) described earlier. NPS pollutant loadings projected with WMM are based on annual runoff volumes and storm event mean concentrations (EMCs) for each pollutant type and each land use category. Pollutant loads were projected under both present and future land use conditions using the following scenarios:

- Existing land use with existing BMPs: This scenario is best described as "existing conditions" and will be used in the evaluation as the baseline for comparison.

- Future land use with existing BMPs: This scenario represents the loading from future land uses if no new BMPs are built. When compared with the results from existing land uses in the existing BMPs, this scenario illustrates the increases in loading due to future growth if such growth is not regulated.

- Future land use with existing BMPs and proposed BMPs: This scenario represents the loading for future land uses once the proposed regional wet detention facilities with pretreatment have been constructed. When compared with the results from future land uses without control, this scenario illustrates the reduction in pollutant loading from the implementation of the recommended plan.

The recommended BMP Treatment Train is discussed in the previous section titled "Recommended Best Management Practices." The removal efficiencies composite of retention swales and wet detention is based on the average annual runoff volume capture estimated with STORM.

Future Land Use with Recommended BMPs

As discussed earlier, a BMP treatment train is recommended for the future development in the Lake Hart basin in order to minimize water quality impacts. The primary structural controls are 0.25 inch of pretreatment swale retention volume in series with regional wet detention ponds. Removal efficiencies were calculated for these BMPs in series based on primarily a volumetric reduction from the retention plus an additional removal of the remaining pollutants from the wet detention ponds. Combined removal efficiencies were projected to range from 72% for TKN to 96% for TSS. The average annual and seasonal pollutant loads under existing and future land use (with recommended BMPs) conditions are presented in Table A-11.

Compared to existing loads, future annual nonpoint source oxygen demand loads with the recommended BMPs are projected to increase for BOD and decrease for COD and future annual sediment loadings are projected to decrease or remain approximately the same. BOD loads are projected to be approximately 1.1 times greater than existing loads and COD loads are projected to decrease by approximately 0.9 times. TSS loads under future conditions with the recommended BMPs are projected to be approximately

0.4 times the existing TSS loads and TDS loads are projected to be approximately 0.9 times.

Total average annual nonpoint source nutrient loadings are projected to decrease for one of the four constituents. The other three are projected to decrease only slightly, therefore, remaining virtually the same. Total-P, TKN and NO_2+NO_3 are projected to approximately remain the same. Dissolved-P is projected to be approximately 0.9 times the existing loads.

Annual nonpoint source heavy metal loadings are projected to decrease for one of the four constituents. Only one constituent increases and the other two remain approximately the same. Lead, is projected to be approximately 0.7 times lower. Zinc loadings are projected to be approximately 1.3 times greater. Copper and cadmium remain approximately the same as existing loads.

In summary, five of the 12 constituents are projected to decrease and five are projected to remain the same under future land use conditions with the recommended BMPs. Loadings of two of the constituents are projected to be greater than existing loadings. The constituents projected to increase are BOD and zinc. BOD increases can be controlled by the use of fountains (i.e., oxygenation) in the wet detention ponds. Slight increases in zinc loadings are not expected to be a problem because wetland plants utilize this metal in a beneficial manner. As previously shown, the overall pollutant loadings from future land use conditions with the recommended BMPs suggest that the recommended BMPs will be effective at minimizing future impacts to water quality.

Table A-11. Average Annual Loadings for Existing and Future Land Use Conditions with Recommended Best Management Practices for the Future Condition

Basin: Entire Lake Hart Study Area

Constituent	Existing Land Uses With Existing BMP's			Future Land Uses With Recommended BMP's		
	Wet Season Loads in Surface Runoff (lbs/yr)	Dry Season Loads in Surface Runoff (lbs/yr)	Annual Loads in Surface Runoff (lbs/yr)	Wet Season Loads in Surface Runoff (lbs/yr)	Dry Season Loads in Surface Runoff (lbs/yr)	Annual Loads in Surface Runoff (lbs/yr)
BOD	90,687	69,821	160,508	102,622	79,009	181,631
COD	997,277	767,815	1,765,092	890,577	685,665	1,576,242
TSS	214,771	165,355	380,126	85,860	66,105	151,965
TDS	2,361,045	1,817,796	4,178,841	2,171,423	1,671,803	3,843,226
Total P	3,906	3,007	6,913	3,795	2,921	6,716
Dissolved P	1,916	1,475	3,391	1,658	1,276	2,934
TKN	25,203	19,404	44,608	24,713	19,027	43,741
NO_2+NO_3	7,652	5,891	13,544	7,434	5,724	13,158
Lead	125	96	221	90	69	159
Copper	66	51	116	64	49	113
Zinc	196	151	347	248	191	439
Cadmium	1	1	2	1	1	2
Runooff (ac-ft/yr)	8,529	6,567	15,096	12,372	9,526	21,898
Runoff (in/yr)	14	10	24	20	15	35
% Impervious			41			68
Basin Area (acres)			7,578			7,578

Water Quantity Results

Introduction

The driving force behind the need for the Lake Hart Basin MSMP was the City's desire to identify stormwater infrastructure needs in this urbanizing basin. Infrastructure needs include improvements necessary to resolve existing problems in the PSWMS as well as avoid potential problems resulting from proposed development. In this study area includes over 4,500 acres of developable property. In terms of water quantity, problems may be in the form of building or road flooding or areas with excessive velocities that could cause significant erosion. For these types of analyses, stormwater model calibration is valuable. Model calibration is essentially a "reality check" to show that the modeled system adequately represents the actual system.

Once SWMM was calibrated, it was used in this plan to identify current levels of service (LOS) and infrastructure needs to accomplish the desired LOS. This was done by comparing peak flood stages from the model results with known critical elevations, such as top-of-road elevations, and any resulting overtopping was compared to the desired level of service for the determination of potential flooding problems and infrastructure or ordinance needs. Likewise, peak velocities in each element in the system were compared to threshold values for the determination of potential excessive velocity problems. Another important element of this study was establishing PSWMS flood stages under future land use conditions and existing hydraulic conditions. Existing and future flood stages are important for guiding future development and determining the relative

Model Calibration

Model calibration refers to the adjustment of model parameters so that the model results (e.g. peak water surface elevations) are in reasonable agreement with a set of observed data. A reasonable range of values for the adjustment of parameters is established through review of the hydrologic literature, and adjustments outside of those ranges are only made if some unusual hydrologic condition exists. The model is considered well-calibrated when it is in reasonable agreement with the data for a comparable independent event without any model adjustments. This process is called model verification. Calibration and verification are desirable to establish a "reality check" of predicted stages, flows, and velocities.

The two primary data requirements for model calibration are gauged rainfall and runoff for the study area. When selecting a calibration storm, the rainfall and runoff data must be sufficiently documented in appropriate time intervals so that variations in rainfall intensity and the associated runoff can be described. Data should be recently acquired so that the current conditions existing in the study area are accurately represented. Additionally, to account for the spatial distribution inherent in Florida rainfall, data should be available at various rainfall stations throughout the study area.

For this study, three rainfall stations were identified within one mile of the study area (Boggy Creek rain gauge, Lake Hart rain gauge, and the Orlando International Airport rain gauge). These three stations record rainfall data on a continuous basis. Because of their proximity to the study area, they were considered to be acceptable for use in model calibration. The data collection phase of the Lake Hart Basin MSMP revealed that flow data were not available for any site in the study area and stage data were limited.

Based on the available data, a normal water surface elevation of 77.0 feet-NGVD was selected as a initial condition in the stormwater model for Lake Nona, Red Lake, and Buck Lake. The normal water surface elevation presented in the Orange County Lake Index Report (77.6 feet-NGVD) was reduced based on the historical measurements obtained from Orange County.

A sensitivity analysis was performed to determine the influence the normal water surface elevation has on the simulated peak water surface elevations in Lake Nona, Red Lake, and Buck Lake. The normal water surface elevations selected for the three lakes were 75.5 ft-NGVD for the low end of the range (known invert elevation of discharge point) and 77.6 ft-NGVD for the upper end of the range (normal water surface elevation reported by Orange County). Using these ranges, the 100-year/72-hour design storm event was simulated for existing land use conditions. The resulting peak water surface elevation ranges were 78.3 to 80.0 ft-NGVD for Lake Nona and 79.4 to 80.1 ft.-NGVD for both Red Lake and Buck Lake.

Using the selected normal water surface elevation of 77.0 ft-NGVD, the simulated 25-year/24-hour peak water surface elevations for Lake Nona, Red Lake, and Buck Lake (from this study) were 78.5, 78.8, and 78.8 ft-NGVD, respectively. This is within 0.2 feet of the 25-year/24-hour peak water surface elevation for Lake Nona and within 0.1 feet of the 25-year/24-hour peak water surface elevations for Red Lake and Buck Lake obtained from the Lake Nona conceptual permit issued by the SFWMD.

Level of Service and Problem Area Definitions

For the 100-year/72-hour design storm event, the simulated peak water surface elevations were 79.5, 79.7, and 79.7 ft-NGVD for Lake Nona, Red Lake, and Buck Lake, respectively. For Lake Nona, the simulated 100-year/72-hour peak water surface elevation is 0.1 feet less than the 100-year peak water surface elevation obtained from FEMA. For Red Lake and Buck Lake, the 100-year/72-hour peak water surface elevation simulated as part of this study is 0.1 feet more than the 100-year peak water surface elevation reported by FEMA. A summary of these comparisons is presented in Table A-12. Based on the results of this comparison, the model was considered calibrated for master planning purposes.

Table A-12. Comparison of Reported and Simulated Peak Surface Water Elevations

Location Model Node		25-Year Design Storm			100-Year Design Storm		
		SFWMD 1994 Permit (ft-NGVD)	CDM 1996 (ft-NGVD)	Elevation Difference (ft-NGVD)	FEMA 1989 (ft-NGVD)	CDM 1996 (ft-NGVD)	Elevation Difference (ft-NGVD)
Lake Nona	10930	78.7	78.5	-0.2	79.6	79.5	-0.1
Red Lake	10870	78.7	78.8	-0.1	79.6	79.7	0.1
Buck Lake	10830	78.7	78.8	-0.1	79.6	79.7	0.1

Water Quantity Evaluation of Existing PSWMS

The PSWMS for the Lake Hart Basin was modeled in RUNOFF and EXTRAN to determine and quantify potential problem areas under existing and future land use conditions, using the 2-, 10-, and 25-year /24-hour design storm events and the 100-year/72-hour design storm event. As appropriate for master planning, existing structures within the PSMS were assumed to be in a maintained condition. This maintenance is costed and summarized in the "Recommendations" section of this appendix. It is also important to understand what a frequency of a design storm (e.g., 25-year frequency) event implies. A 25-year frequency does not mean that the rainfall event will occur once every 25 years. A 25-year frequency means the event has a 4% (1 in 25) chance of occurring or being exceeded in any given year.

Resultant flood stages in the PSWMS were developed for the existing and future land use scenarios. Increases in depth from existing to future land use conditions range from approximately 0.0 ft to 0.4 ft. The relatively small increases in stage, despite the increases in imperviousness, are a result of two conditions. First, the PSWMS has a very large storage capacity in the lakes and wetlands with very flat floodplains, so increases in flow rates will not cause large increases in stage. Second, because the seasonal high groundwater table is close to the surface over much of the study area (limited soil storage capacity), the decrease in pervious area from present to future land use conditions does not result in a large loss of storage in the soil column. The high groundwater table causes the pervious areas of the basin to effectively become impervious after minimal rainfall.

Therefore, regulating floodplain storage and floodway conveyance in this basin, along with the regional wet detention ponds and identified capital improvements, is important.

Based on the level of service criteria previously discussed, deficiencies in the PSWMS were:

- Problem P-1 is the flooding of Narcoossee Road by 0.3 feet during the two-year design/24-hour storm event and by as much as 1.2 feet during the 100-year/72-hour design storm event (model node 10895). This problem is caused by the tailwater condition established for node 10905 from Orange County stage data, field inspection, and 1 foot photogrammetry. The location of this problem area is shown on Figure A-15.

- The peak simulated velocities for in the PSWMS elements are presented in Table A-13 for the two-year and 10-year events under future land use conditions. High velocities for lower return period events are an indicator of potentially excessive erosion which can cause structure failure and degrade water quality.

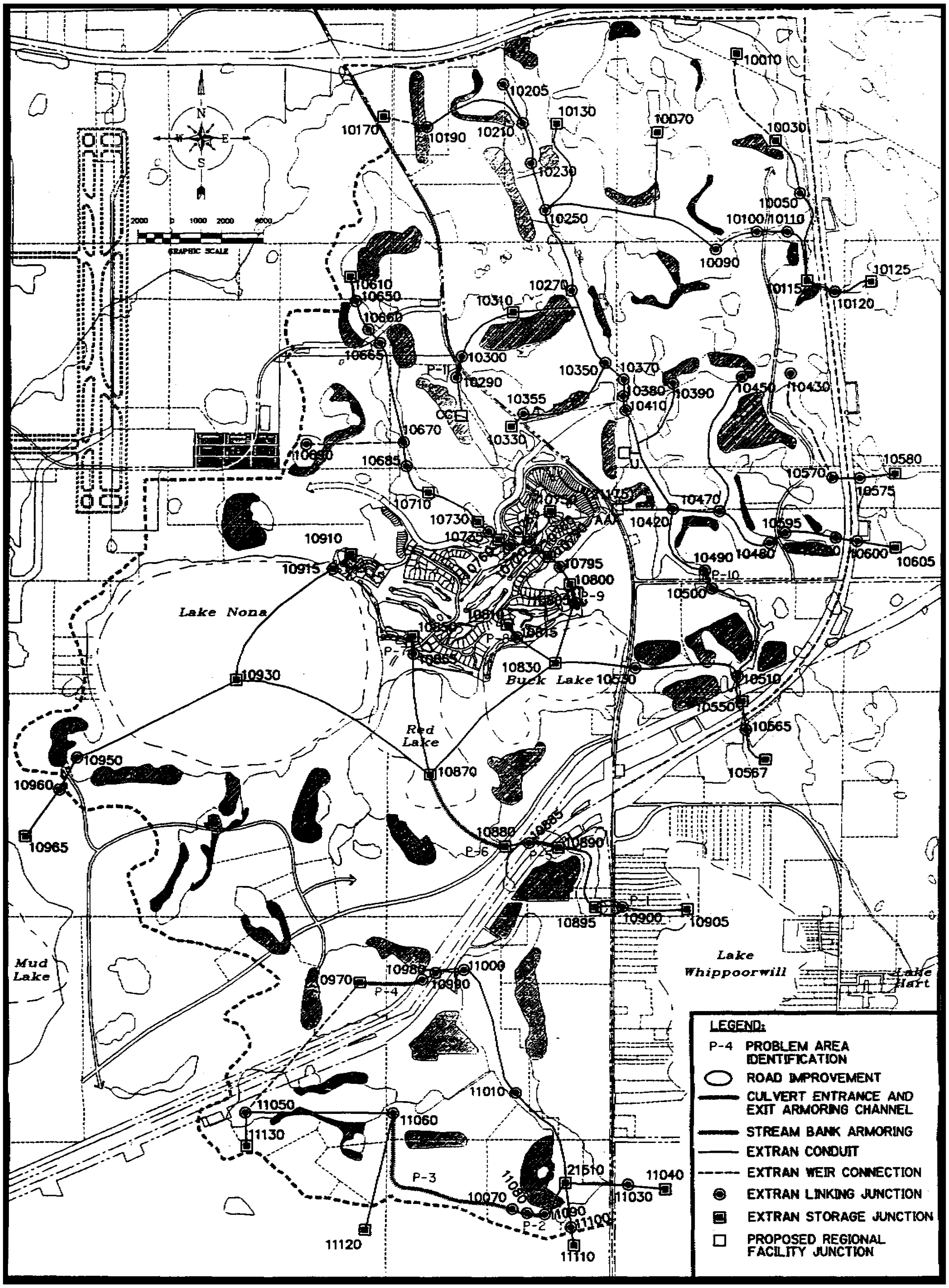


Figure A-15. Problem area identification map (Reprinted courtesy of the City of Orlando, FL).

Table A-13. Excessive Velocity Determination for Future Land Use

Channel ID	Channel Type (1)	2-Year Event (2)	10-Year Event (2)	Problem ID(3)
11080	C		2	P-2
11060	N		1	P-3
10970	C		2	P-4
10885	C		2	P-5
10870	C	2	2	P-6
10851	C	2	2	P-7
10811	C	2	2	P-8
10801	C	2	2	P-9
10492	C	2	2	P10
10491	C	2	2	P-10
10290	C		2	P-11

(1) Channel Type: C = culvert, bridge, storm sewer, or paved channel. N = natural earthen channel.
(2) Problem Type: 1 = Natural channel velocity > 3ft/sec. 2 = Culvert, bridge, sewer, or channel velocity > 7 ft/sec.
(3) Velocity problem areas have been assigned Ids.

Proposed Regional Wet Detention Facilities

The siting of the proposed regional wet detention facilities was accomplished through a cooperative effort between the City of Orlando and the major property owners in the study area. Through this cooperative work effort, regional facilities were strategically located to meet public, private, and environmental interests to the maximum extent practicable. Through this process, a total of 52 wet detention ponds, nine of which are existing borrow pits, were conceptually designed for this study area. The facilities provide regional flood control and water quality protection associated with urbanization. Conceptually, stormwater runoff would be collected in a pretreatment and conveyance system and delivered to the proposed regional facility, treated (via wet detention), attenuated for peak flow and velocity, and discharged into the PSWMS through a V-notch weir/swale spreader system.

A conceptual plan view of a proposed facility is presented in Figure A-16. As can be seen in the figure, the proposed regional facilities were located along existing wetlands in an elongated manner. The wet detention facilities can also provide other benefits such as waterfront property, potential recreational areas, and hydrate wetlands thus protecting them from potential development impacts.

The locations of the proposed regional wet detention facilities in the study area are presented on Figure A-17. The facility footprints shown on the figure represent the 100-year/72-hour peak water surface elevation predicted to occur at each site using the stormwater model developed for this study.

Use of Existing Borrow Pits as Stormwater Facilities

Existing waterbodies may be used for detention purposes as long as the SFWMD grading criteria pertaining to ponds or lakes near wetlands are met (Section 4.10 of the SFWMD MSSW Permit Application Manual Volume IV). Additionally, the SFWMD requires that side slopes be no steeper than 4:1 to a depth of two feet below the control elevation. Existing borrow pit acreage within the study area and, if necessary, increased surface area requirements are presented in Table A-14. As previously stated, there are nine existing borrow pits identified as potential regional wet detention facilities. These include potential sites P, V, RR, TT, UU, VV, SS, ZZ, and WW shown on Figure A-18.

Flood Control Benefits

The proposed regional facilities were evaluated using SWMM for each design storm event under future land use conditions. The resulting peak water surface elevations were determined from the hydraulic analyses. The elevations are compared to existing and future land use conditions without the proposed regional facilities. The simulated peak water surface elevation for the 2-, 10-, 25-year/24-hour design storm events and the 100-year/72-hour design storm event under future land use conditions with the proposed regional facilities are less than or equal to the simulated peak water surface elevations under existing land use conditions at almost every point within the study area.

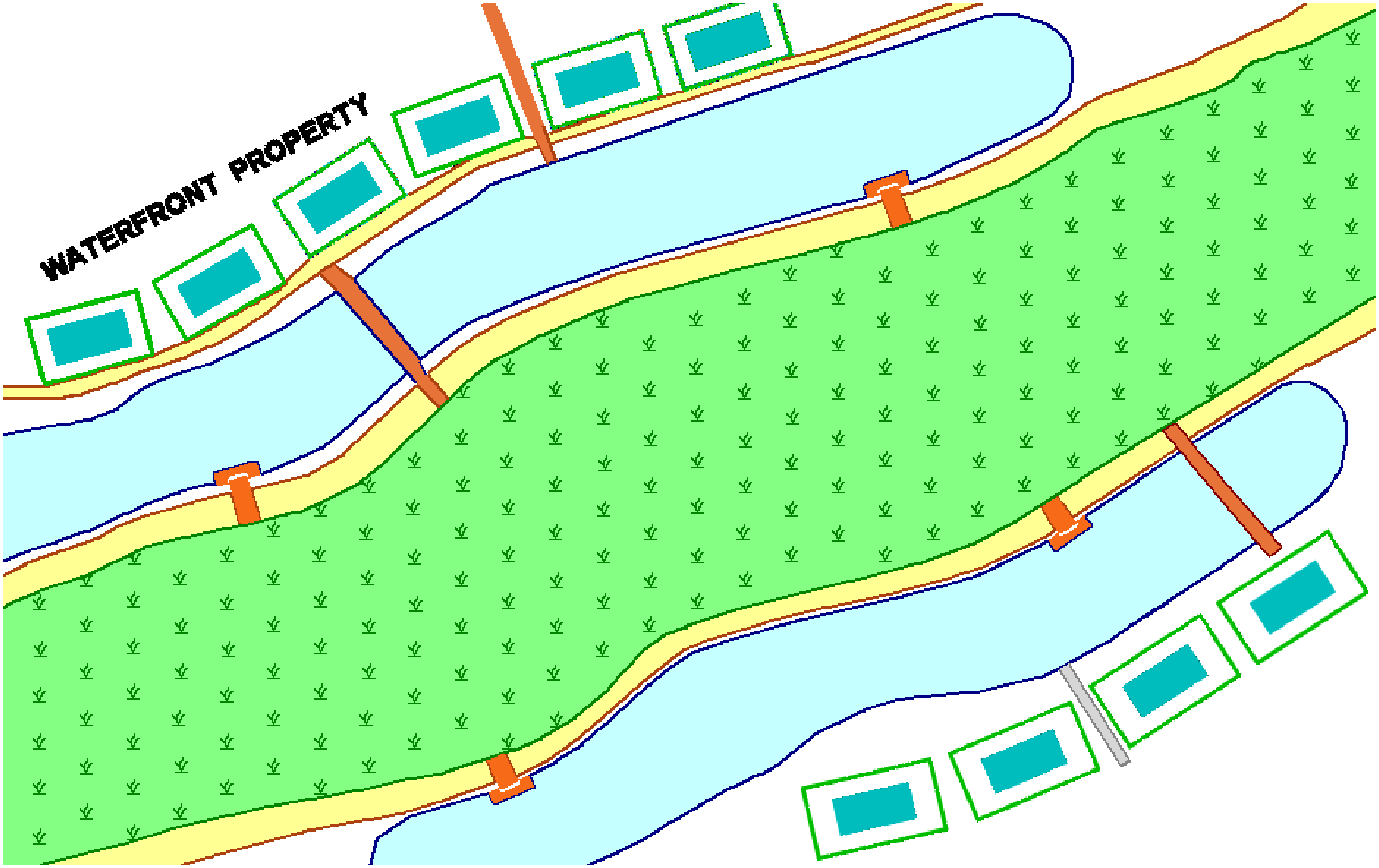


Figure A-16. Typical wetlands and ponds layout (Reprinted courtesy of the City of Orlando, FL).

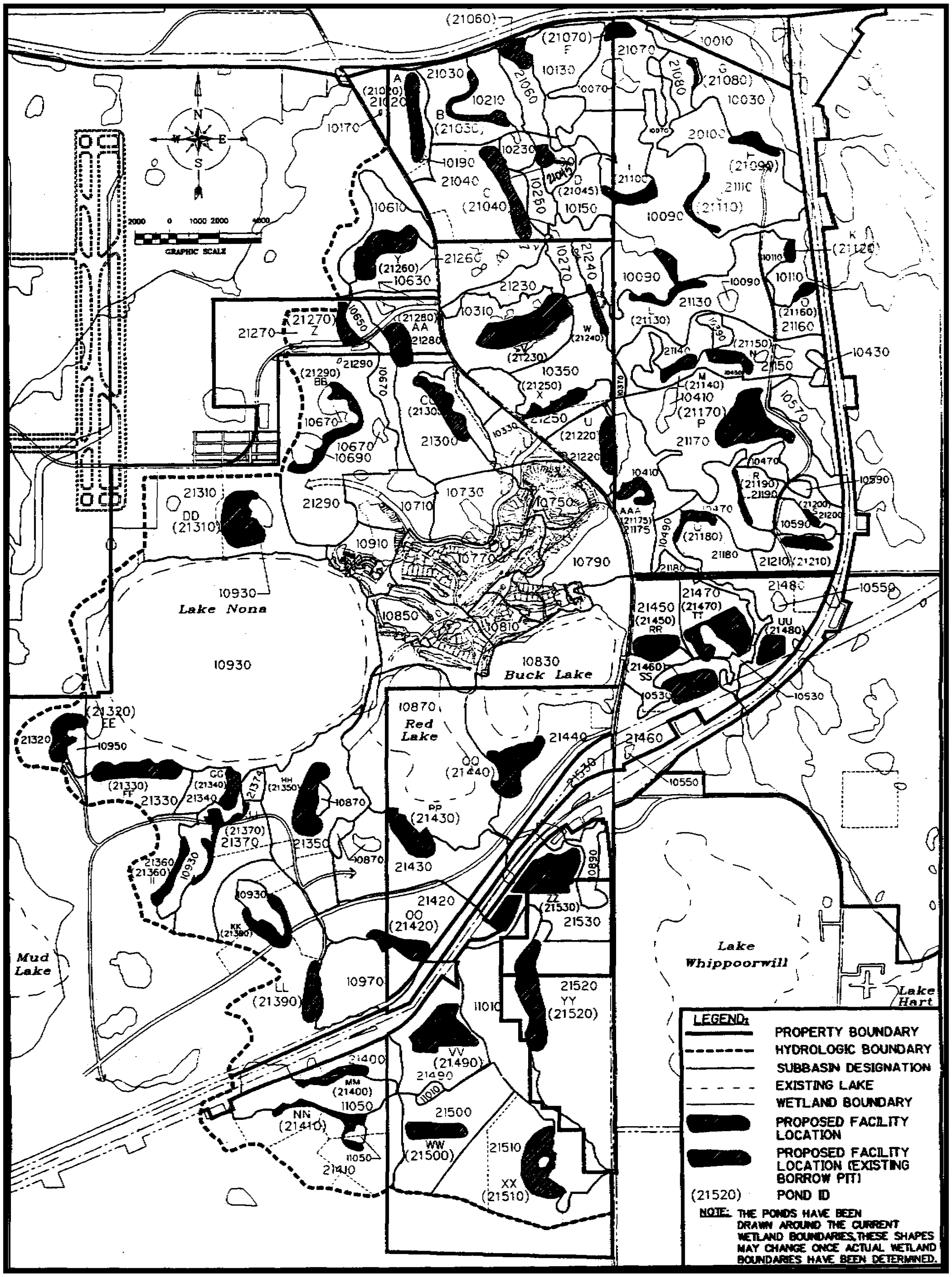


Figure A-17. Proposed regional wet detention facilities (Reprinted courtesy of the City of Orlando, FL).

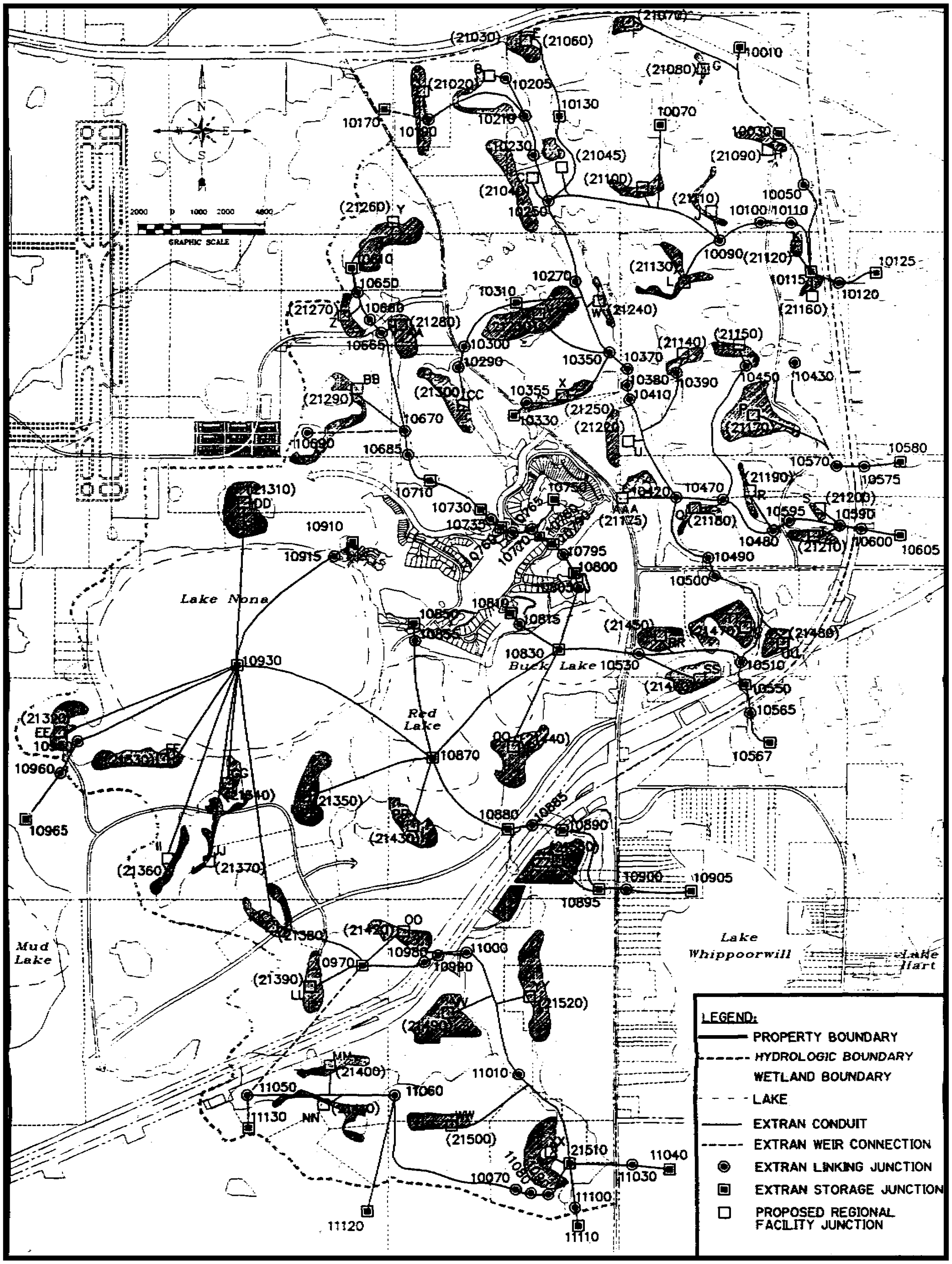


Figure A-18. Alternative PSWMS nodal schematic (Reprinted courtesy of the City of Orlando, FL).

Table A-14. Changes in Surface Area of Sites Currently Existing as Borrow Pits

Pond Node ID	Existing Surface Area of Borrow Pit (Acres)	Required Surface Area for 100-YR (Acres)	Increase in Surface Area of Borrow Pits (Acres)
21170 (P)	33	34	1
21230 (V)	32	33	1
21450 (RR)	6	12	6
21460 (SS)	12	15	3
21470 (TT)	22	22	0[1]
21480 (UU)	9	15	6
21490 (VV)	26	27	1
21500 (WW)	5	12	7
21530 (ZZ)	36	37	1

1. The existing surface area is greater than what is required. Therefore, no increase in the surface area of the existing site is necessary.

Peak flows at the discharge points of the study area were also compared to show that downstream (Orange County) peak flows and peak water surface elevations are controlled under post-development conditions. With the proposed facilities, significant flow rate reductions are obtained when compared to flow rates simulated under future land use conditions without the regional facilities. The predicted flow reductions obtained by incorporating the proposed facilities into the PSWMS are also below those predicted at the discharge points from the study area under existing land use conditions. This analysis shows that the proposed regional wet detention facilities are effective in providing flood control for future development.

Recommendations

Introduction

A summary of the recommendations for the Lake Hart basin MSMP is provided in this section. The Capital Improvements Program (CIP) is outlined along with operation and maintenance considerations, nonstructural controls, and stormwater monitoring.

Capital Improvement Program for Structural Controls

Review of Factors

As previously discussed, six major factors were considered in the formulation of the CIP program recommendations. These factors are:

1. Technical feasibility and reliability
2. System maintainability
3. Sociopolitical acceptability
4. Economics
5. Environmental consistency
6. Financial ability

Technical Feasibility and Reliability

The recommendations have been formulated to be feasible and reliable from a technical standpoint. Flooding problems are solved within the level of service guidelines defined for this study and cost-effective water quality control is provided (pretreatment and wet detention). Conveyance solutions are all gravity-driven and regional storage of water (swales, ponds) is proposed as needed for proposed development and the Narcoossee Road Improvement Project.

System Maintainability

The proposed project needs to address operation and maintenance (O&M) issues. For example, the proposed regional approach promotes the need for fewer stormwater management facilities compared to the onsite approach which requires many ponds to achieve the same level of service. The larger regional facilities are more likely to be maintained on a regular basis.

Sociopolitical Acceptability

The recommendations address flooding and water quality concerns and are consistent with existing regulations. Public information may become an important aspect of the recommendations in the future since improved watershed protection can be achieved though public education and involvement. The recommended plan reduces nonpoint loads to the lakes, maintains or lowers existing flood stages, and does not adversely impact healthy wetlands which are a large component of the PSWMS.

Additionally, because the Lake Hart MSMP serves City, public, and private developer interests, the project needed to be conducted cooperatively between interested parties to the extent practicable. This was accomplished through coordination meetings with City staff, regulatory agency staff, and private developers.

Economics

The recommended plan provides sound technical, environmental, and social benefits, as well as providing for the most cost-effective water quantity and water quality controls. The recommendations appear to be cost-effective for joint private/public funding partnership of stormwater management capital improvement projects as development occurs.

Environmental Consistency

The recommendations have been formulated to minimize wetland impacts and to promote aquifer recharge, where possible. No ponds or BMPs were sited in known wetlands.

Financial Ability

An important consideration in this project is the ability to fund the recommended plan. Funding of the regional facilities will likely be a public/private venture. The project needs to have a reasonable chance of being funded without causing financial hardship. Because of the large number of recommended regional facilities, phasing of capital improvements will be concurrent with the development phasing in the basin.

CIP Summary

Based on these six criteria, 52 regional wet detention facilities (nine are modified existing borrow pits) are recommended for the Lake Hart basin. Each facility would serve a dual purpose of flood control and water quality protection. The location of each facility reflects the cooperative siting efforts between the City and private land owners. Because of the high groundwater table in the study area, it is recommended that pretreatment be provided (0.25 inches) upstream of each facility instead of the retention requirements for wet detention facilities in OUSWMM. The pretreatment requirement is considered to be applied innovative technology for the basin and is viewed as an enhancement to OUSWMM.

In addition to the proposed regional facilities, it is recommended that the Narcoossee Road (Problem P-1 at model node 10895) crossing of the tributary flowing southward from Red Lake to Lake Whippoorwill be raised to an elevation above the 25-year/24-hour designs storm event under future land use conditions with the proposed regional facilities in place (77.8 ft-NGVD).

Based on the results of the December 5, 1995 field inspection, it is also recommended that the culvert and conveyance channel under the dirt road just downstream of Red Lake be restored. The culvert and approach channel appeared to be in poor condition from cattle traffic.

Excessive velocities were identified in 11 conduits in the basin. All but one of the conduits (11060) is a culvert pipe. Conduit 11060 is an excavated drainage canal. For this canal, visual inspection for erosion problems should be made and where erosion is evident the bank should be stabilized. For the closed conduits (culvert crossings), channel bank and bottom armoring is recommended for a distance of 30 feet upstream and downstream of the culvert crossing. Three of the culverts with high velocities are associated with outlet works from existing facilities within the Lake Nona development

(Model nodes 10850, 10810, and 10800). Armoring downstream of these structures should be done as part of these capital improvements.

A map showing the overall recommended CIP plan is presented in Figure A-19. CIP planning level costs for these improvements are summarized in Table A-15.

Project Phasing

Phasing of capital improvements was based on scheduled and planned construction projects. The first planned change in the basin is the City's Narcoossee Road Improvement Project scheduled for construction in 1997. In order to address stormwater management for this project, the proposed regional facilities that can serve both new development and Narcoossee Road are going to be constructed first. The City will develop a cost sharing plan with private development for these dual purpose facilities. The first phase of pond construction will serve Narcoossee Road (funded by City). Private land owners can then expand these facilities as development occurs.

The remaining facilities should be built as development plans are approved and scheduled for construction. The City plans to use the stormwater model developed for this Lake Hart basin MSMP to identify which facilities will be needed for each new development. The phasing of these structures will require coordination between City staff and land developers planning to build within the basin.

Operation and Maintenance

Operation and maintenance are critical elements of the MSMP. Control measures that are not maintainable provide short-lived, expensive solutions. Additionally, stormwater management systems that are not adequately maintained cannot be relied upon to provide the desired levels of service. The control measures recommended were developed with consideration of maintenance issues. For example, forebays have been recommended for all regional wet detention facilities to reduce the maintenance requirements and extend the effectiveness of the facilities. The City is considering taking over the operation and maintenance responsibility for the regional facilities constructed under a cost sharing program. The City would fund the cost of the operation and maintenance through their existing stormwater utility.

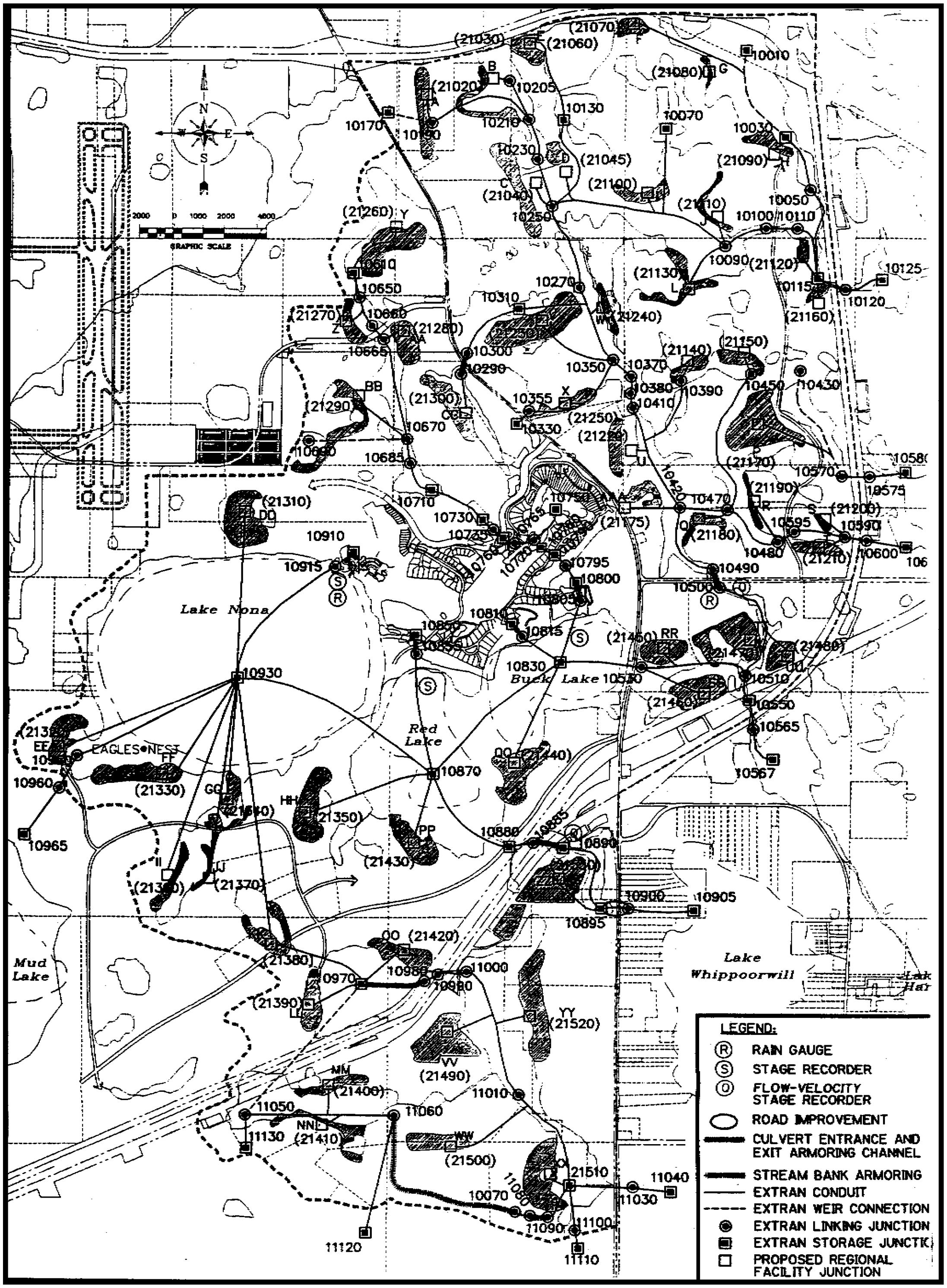


Figure A-19. Capital Improvements Plan Map (Reprinted courtesy of the City of Orlando, FL).

Table A-15. Conceptual Capital Cost Estimate for Lake Hart Basin Southeast Annexation Area

	Pond ID	Capital Cost ($)
City Ponds	21250 (X)	984,000
	21260 (Y)	2,234,000
	21300 (CC)	1,485,000
	21450 (RR)	662,000
	21175 (AAA)	521,000
Subtotal		5,886,000
Developer Ponds	21020 (A)	1,133,000
	21030 (B)	764,000
	21040 ©	1,456,000
	21045 (D)	325,000
	21060 (E)	644,000
	21040 (F)	430,000
	21080 (G)	150,000
	21090 (H)	545,000
	21100 (I)	634,000
	21110 (J)	400,000
	21120 (K)	195,000
	21130 (L)	951,000
	21140 (M)	447,000
	21150 (N)	591,000
	21160 (O)	241,000
	21170 (P)	165,000
	21180 (Q)	447,000
	21190 (R)	272,000
	21200 (S)	150,000
	21210 (T)	545,000
	21220 (U)	582,000
	21230 (V)	190,000
	21240 (W)	371,000
	21270 (Z)	529,000
	21280 (AA)	899,000
	21290 (BB)	1,320,000
	21310 (DD)	1,786,000
	21320 (EE)	1,035,000
	21330 (FF)	1,425,000
	21340 (GG)	560,000
	21350 (HH)	1,583,000
	21360 (II)	605,000
	21370 (JJ)	651,000
	21380 (KK)	1,035,000
	21390 (LL)	885,000
	24100 (MM)	771,000
	21410 (NN)	1,674,000
	21420 (OO)	945,000
	21430 (PP)	1,200,000
	21440 (QQ)	1,771,000
	21460 (SS)	189,000
	21470 (TT)	182,000
	21480 (UU)	470,000

Table A-15. Continued.

	Pond ID	Capital Cost ($)
	21490 (VV)	119,000
	21500 (WW)	589,000
	21510 (XX)	1,816,000
	21520 (YY)	1,861,000
	21530 (ZZ)	182,000
Subtotal		35,710,000
Channel Armoring Ponds	P-2	13,000
	P-3	33,000
	P-4	13,000
	P-5	13,000
	P-10	76,000
	P-11	13,000
Subtotal		161,000
Total		41,757,000

1 City pond capital costs include $15,000/acre for land acquisition (land acquisition costs are not included in developer pond costs).
2
3 Capital costs are for stormwater related facilities only and do not include stormwater related utility rehabilitation and replacement.
4
5 Costs are in 1996 dollars.
6
7 These costs include a 40% contingency for engineering, surveying, permitting, and contractor's overhead and profit as well as mobilization and standard contingencies.
8
9 Excavation costs may be reduced by the use or sale of fill material.
10
11 Field verification of problem areas is recommended prior to channel armoring.

Annual operation and maintenance costs are summarized in Table A-16. These costs include the costs associated with maintaining the existing facilities and recommended control measures.

Table A-16. Annual Operation and Maintenance Cost Summary for Lake Hart Basin Southeast Annexation Area

Item	Cost ($/yr.)
1) Maintain 53 regional facilities. This includes labor and equipment to provide annual grounds maintenance and inspection of control structures, channels, silt levels, erosion, and vegetation. Also included are three mowings per year and removal of excess silt and Vegetation every five to seven years.	424,000
2) Maintain 33 bridges/culverts within the primary stormwater management system (once every two years with annual inspection).	33,000
TOTAL ANNUAL OPERATION AND MAINTENANCE COSTS	457,000

1. Routine maintenance of natural channels was not considered since the majority of the PSWMS consists of natural wetlands.
2. Maintenance of channels for a distance of 50 ft. upstream and downstream of culverts is included in culvert maintenance costs.
3. Problem ID P-6, reach 10870, is a small trail crossing which should be maintained if an erosion problem is identified from field inspection.

Nonstructural Controls

Nonstructural controls were considered to help control both water quantity and water quality aspects of stormwater. Nonstructural controls are not constructed capital projects but rather are source controls, ordinances, and regulations that depend on participation by municipalities and residents to minimize the water quantity and quality impacts associated with development. A summary of recommended nonstructural controls follows:

1. Public information program
2. Fertilizer application control
3. Pesticide and herbicide control
4. Solid waste management and control of illegal dumping
5. Directly connected impervious area (DCIA) minimization
6. Water conservation landscaping
7. Illicit connections - identification and removal
8. Erosion and sediment control on construction sites
9. Stormwater management ordinance requirements
10. Stormwater management system maintenance

The following provisions are recommended to supplement the existing OUSWMM

1. 100-Year Floodplain Protection: This provision already exists in OUSWMM, but

2. because of its importance in preventing future flooding, it is re-emphasized in this section of the report. To assure proper flood hazard management, it is recommended that compensating storage be required for all construction, development, or site alteration so that existing 100-year floodplain storage in the City is maintained; and therefore, flood stages are not increased or moved onto adjacent lands by the development.

3. Aquifer Recharge: Although the potential for aquifer recharge in this basin is low due to the soils and the groundwater table, the overall concept is an important consideration. A general consideration is to retain the first three inches of runoff over the DCIA on SCS Hydrologic Group A soils and two inches of runoff over the DCIA on SCS Hydrologic Group B soils. In addition, it is recommended that swale pretreatment for these areas be provided to increase the amount of soil treatment before discharge into the aquifer.

4. First-Floor Elevations: Variances to construct dwelling first-floor elevations below the 100-year floodplain should not be allowed or variances should be deed-recorded with sale of the property. Variances encourage people to build in flood prone areas around lakes and streams. It is inevitable that these dwellings will eventually be flooded. This can cause public pressure on the City to drain wetlands and regulate or drain lakes -- a policy that is inconsistent with fishery habitat, aquifer recharge, and water quality.

5. Floodway Management: SFWMD allows the filling of a floodway as long as it does not cause more than a one-foot increase in the flood stage within the floodway (Federal Emergency Management Agency standard). This can have a severe cumulative impact on property in or adjacent to the floodway farther downstream. It is recommended that floodway encroachment be prohibited. It is recommended that no net encroachment be allowed within the future land use top-width-in-flow for the 100-year storm.

6. Water Quality: It is recommended that the City continue to require water quality performance standards as outlined in Chapter 40, Florida Administrative Code, that are based upon receiving water classifications, until more detailed watershed specific data are known from monitoring and/or state water policy mandates from the Florida legislature occur.

7. Reuse: The conservation of water resources is increasingly encouraged where it is applicable. The use of landscaped swales is recommended to promote reuse of some of the stormwater runoff.

Monitoring

A comprehensive monitoring program includes many facets of data collection and is used to accurately define the hydrologic and hydraulic characteristics of a watershed. This report recommends that the City augment existing monitoring data with an overall program in order to provide additional data necessary to evaluate the stormwater quantity and quality of the Lake Hart basin. The monitoring program should address the following:

1. Identification of rainfall and flow/stage data at key points of interest to calibrate and verify model analysis tools.

2. Current status of water quality including ambient data, dry weather flow from stormwater outfalls, and wet weather runoff as event mean concentration (EMC) values for land use types.

3. Trends in water quality due to land use changes and BMP implementation.

4. Regulatory assistance with state and federal permitting.

5. Compliance monitoring to document permit compliance.

The City can benefit from a monitoring program that addresses the preceding. A monitoring program will support implementation of the Lake Hart basin MSMP and the NPDES MS4 program. The overall monitoring program recommended for the City is described below.

Recommended Monitoring Program

Rainfall

This plan recommends that the City supplement the existing rainfall stations operated and maintained by Orange County and NOAA (airport rain gauge) with two stations. One would be combined with the stage recorder proposed for Lake Nona and the other would be combined with the flow-velocity recorder proposed at Moss Park Road. These rainfall stations should record rainfall data at a minimum of 15-minute intervals. The general locations of these stations are presented in Figure A-18.

Water Quality

It is recommended that the City maintain the ambient water quality monitoring program conducted by Orange County for Lake Nona, Red Lake, and Buck Lake as to further document the long-term water quality.

Water Quantity

The City should consider a joint effort with USGS to establish a stream gauge monitoring program for the Lake Hart basin. Daily stages should be recorded for Lake

Nona, Red Lake, and Buck Lake. Stations that measure flow and velocity are also recommended on the downstream side of Moss Park Road (model node 10500), the downstream side of Narcoossee Road (flows from Buck Lake, model node 10530), and on the downstream side of the Central Florida Greenway (flows from Red Lake to Lake Whippoorwill, model node 10890). Stream gauges at these locations will help the City monitor flow from the major tributaries that outfall into Orange County. It is recommended that the City propose that USGS establish, operate, and maintain the gauge and data. The locations of these facilities are also presented on Figure A-18.

Mosquito Control

As part of the evaluation of various alternatives, it is recommended that the City consider the potential for mosquito breeding. Some minor modifications and considerations in the design of various BMPs are needed to minimize the breeding of mosquitoes. The primary concern is stagnant water, which provides a breeding ground for mosquito larvae. Water that stands for periods of greater than 72 hours provides a suitable environment for the breeding of mosquito larvae.

To effectively control mosquitoes, it is suggested that the following guidelines be considered for the design of BMPs in the Lake Hart basin:

1. Use only Hydrologic Group A soils (or well drained Hydrologic Group B or C soils, water table at least one to two feet below grade) for retention type facilities (e.g., shallow grassed swales). It is suggested that seasonal high groundwater tables and soils be tested for each area on a case-by-case basis to verify that complete storage recovery will occur within 72 hours

2. For wet ponds, use a minimum depth of greater than 18 inches so that minnows can be sustained. Additionally, maintain vegetative density low enough for minnows to access (minnows feed on mosquito larvae)

3. When developing a site for a detention or infiltration pond, use a minimum of 20 feet for the buffer/maintenance strip.

Data Sources and Bibliography

Referenced reports, studies, digital data, and maps were obtained and reviewed for this study. This section is intended to be a data bibliography which lists the sources and types of data used. The following references were evaluated for potential applicability to this Lake Hart MSMP.

- 1993 Annual Report, Orange County Environmental Protection Department, 1993.

- Orange County, Environmental Protection Department, 1993 Lake Ranking for Orange County Lakes by Trophic State Index, by (April 1994).

- 1994 Orange County lake ranking by tropic state index, Orange County Environmental Protection Department, 1995.

- Aerial (color) photogrammetry maps by Belt Collins, FL from Lake Nona Corporation (2.5 inches = 1 mile and 2.33 inches = 1 mile, March 1994).

- Aerial photogrammetry maps for Lake Hart-Lake Mary Jane Drainage Basin with 1 foot contours from Orange County, Florida (1 inches = 200 feet, 1985).

- Aerial photogrammetry maps from Orange County, FL (1 inch = 300 feet, 1990).

- Applications for Development Approval for Developments of Regional Impact (DRIs) for Lake Nona, Lake Hart, St. James Park, and Campus Crusade.

- Basis of Review for Environmental Resource Permit Applications with the South Florida Water Management District (August 1995).

- Brunetti Bal Bay Tract Concept Plan prepared by Berryman and Henigar
- (1 inch = 600 feet, August 1994).

- City of Orlando Engineering Standards Manual Second Edition from the Public Works Department (June 1993).

- City of Orlando Florida Southeast Annexation Area Lake Hart Basin Master Stormwater Plan, February 1996, prepared by Camp Dresser & McKee Inc. and WBQ Design & Engineering, Inc.

- City of Orlando Florida Southeast Annexation Stormwater Management Needs Assessment, June 1995, prepared by Camp Dresser & McKee Inc. This report was the first phase of the Lake Hart MSMP.

- Digital FEMA MAP of the Lake Hart Study Area from the City of Orlando, FL.

- Digital soils file of the Lake Hart area from the City of Orlando, FL.

- Eastern Beltway - Bee Line Interchange Plans from the Orlando-Orange County Expressway Authority.

- Eastern Beltway roadway and drainage as-built plans from the Orlando-Orange County Expressway Authority.

- Eastern Beltway roadway and drainage plans from the Orlando-Orange County Expressway Authority (Sections 454, 455, and 457).

- Existing Drainage Map of Randall/Johnson Trust Property from Miller-Sellen Associates, Inc.

- Existing Survey in the Lake Hart Area. This survey was completed for the Boggy Creek watershed study which includes cross-sections between Lakes Nona, Red and Buck and of the Myrtle Bay Area.

- Existing Survey in the Lake Hart Area from Transportation Engineering, Inc. (1995).

- Existing Survey in the Lake Hart Area computed by DeGrove Surveyors from FEMA (1992).

- FEMA; FIS for the Unincorporated Area in Orange County, FL (December 8, 1989).

- Flood Insurance Rate Maps from Federal Emergency Management Agency (FEMA) (Panels: 400, 425, 550 and 575).

- Future Development Plan for Randall/Johnson Trust from Miller-Sellen Associates, Inc.

- Greendale Master Plan prepared by Davis and Associates (1" = 300', May 1994).

- Growth Management Plan Southeast Annexation Study approved October 17, 1994 from the City of Orlando, FL..

- Lake Hart Master Plan Development Plan from Post, Buckley, Schuh and Jernigan (1 inch = 1333 feet, 1994).

- Lake Nona Application for Conceptual Approval Surface Water Management Permit with the South Florida Water Management District prepared by Miller and Einhouse, Inc. from Lake Nona Corporation (October 1988).

- Lake Nona Construction Plans and as-builts for stormwater facilities provided by Lake Nona Corporation.

- Lake Nona Master Drainage Plan for Phase 1-A (1 inch = 300 feet, December 1988).

- Lake Nona Preliminary Master Plan 6 Future Development Plan prepared by Belt Collins, Florida from Lake Nona Corporation (1" = 1000', September 1994).

- Lake Nona Preliminary Master Plan 6 Future Development Plan prepared by Belt Collins, Florida from Lake Nona Corporation (1 inch = 1000 feet, March 1995).

- Lake Nona South Existing Conditions Drainage Map prepared by Einhouse and Associates, Inc. from Lake Nona Corporation (1 inch = 600 feet).

- Lake Nona Surface Water Management Permit Modification Application for Conceptual Permit No. 48-00195-S with the South Florida Water Management District prepared by Miller and Einhouse, Inc. from the Lake Nona Corporation.

- La Vina Trust Land Use Plan prepared by Burkett Engineering, Inc. (1 inch = 300 feet, May 1995).

- Master Drainage Plan of Randall/Johnson Trust Property from Miller-Sellen Associates, Inc. (1 inch = 400 feet).

- Miscellaneous Permits in the Southeast Study Area from the South Florida Water Management District.

- Narcoossee Road Construction Plans for the City of Orlando from WBQ Design & Engineering, Inc. (May 1995).

- Narcoossee NW, Narcoossee, St. Cloud North, and Pine Castle Fish and Wildlife Service National Wetland Inventory Maps (1988).

- Narcoossee NW, Narcoossee, St. Cloud North, and Pine Castle USGS Quadrangle Maps 7.5 minute series (photo revised: 1980, 1970, 1987 and 1980, respectively).

- Orange County Future Land Use Maps Series of the Lake Hart Study Area from Orange County, FL (August 1993).

- Orange County Lake Index*, 1995 Report* from Orange County Public Works.

- Orlando/Orange County Joint Planning Area Map from City of Orlando Planning and Development Department (May 1994).

- Orlando Urban Stormwater Management Manual (OUSWMM) prepared by Dyer, Riddle, Mills, and Precourt, Inc. Volume 2 Design Criteria, Second Edition from the City of Orlando, Florida.

- Physical and Chemical Data and Plankton Summaries for Lakes Nona, Red and Buck for the period of record from (1972 - 1994), from Orange County Pollution Control Department.

- Rainfall data for the period of record (1974-1992) at the Orlando-McCoy Airport in Florida, rain gauge from the National Climatic Data Center (NCDC).

- Rainfall data for the period of record (1987-1995) at the Boggy Creek rain gauge and for the period of record (1995) at the Lake Hart rain gauge from the Stormwater Management Department of Orange County, FL.

- Randall/Johnson Trust conceptual approval permit from the South Water Management District (Control Number: 48-00653-S, January 1992).

- Realignment of Dowden Road Plans provided by Busch Properties.

- Seventh International Conference on Urban Storm Drainage, Hannover, Germany, 9-13 September 1996. Proceedings Volume I, II, III.

- Soil Survey of Orange County, FL, 1989. This is a typical United States Department of Agriculture Soil Conservation Service (SCS) soils report that provides various surficial-layer soils information for the County. Total soil storage, infiltration rates, and data on surficial "hard pan" layers were used for this study.

- South Florida Water Management District, Management and Storage of Surface Waters Permit Information Manual, Volume IV (May 1994).

- Southeast/Orlando International Airport Future Growth Center Plan Conceptual Framework from the City of Orlando Planning and Development Department (May 1995).

- Southeast Study Area Map with property owners boundaries from the City of Orlando Planning and Development Department (November 1993).

- Southeast Study Area Map with the property owners proposed roadways and the City of Orlando's preferred roadways from the City of Orlando Planning and Development Department (June 1995).

- Survey completed by Regional Engineers, Planners and Surveyors, Inc. (REPS) for use in the Stormwater Modelling (October 1995).

- Upper Kissimmee River Watershed Map of Major Basins from the South Florida Water Management District (SFWMD) (8.5 inches x 11 inches).
- Urban Drainage and Flood Control District, Denver, Colorado, "Urban Storm Drainage Criteria Manual - Volume 3 - Best Management Practices - Stormwater Quality", September 1992.

- Water Quality Data Summary for Lakes Nona, Red, and Buck prepared by Envirosmiths, Inc. (November 1994).